SUMMARY REPORT ON DESIGN GUIDANCE AND ASSESSMENT METHODOLOGIES FOR WAVE SLAM AND GREEN WATER IMPACT LOADING

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1 INTRODUCTION TO THE SAFE-FLOW PROJECT

1.1 Background

Floating production vessels are increasingly being used by the offshore oil and gas industry as a cost-effective, reusable alternative to fixed steel or concrete platforms. The hulls of these vessels often resemble crude oil carriers and indeed are frequently constructed by converting a new or second hand tanker unit. However, they may be sited at a fixed location for a period of years and, consequently, must satisfy very different operational requirements in comparison to those applicable to traditional ocean-going vessels. Purpose built barges are nowadays used regularly as floating production vessels.

Several incidents in recent years have revealed that green water and wave impact loading (which can be grouped under the term 'water impact loading') are important issues for moored floating production systems. Flows of large volumes of seawater across a deck and high velocity impacts of seawater against a vessel's hull, respectively, have caused significant damage to topside equipment and plastic deformation, occasionally penetration, of hull plate. To date, no incident has escalated sufficiently to cause loss of life, serious injury to personnel, environmental pollution, major damage or total loss of the vessel, however, the evident prospect of such an incident is of concern to the offshore industry.

The SAFE-FLOW (SAFE FLOating offshore structures under impact loading of shipped green water and Waves) Joint Industry Project (JIP) was initiated in response to encouragement by the UK Health and Safety Executive (HSE) following earlier joint research, such as the JIP 'F(P)SO Green Water Loading', completed by MARIN in 1997. Its main objective was to investigate 'water impact loading' hazards in detail. The research team included a number of specialist European-based companies/organisations in this field.

Approximately 49% of the SAFE-FLOW research budget is provided by the European Community under the 'Competitive and Sustainable Growth' Programme (1998-2002), while the other 51% is provided by industrial sponsors including oil companies, ship builders, classification societies and consultants.

This document is the summary level report for the SAFEFLOW project: its purpose to provide background, design guidance and information about how to use the data and software developed on the project. Full-scale and model test data, and detailed descriptions, instructions and examples of the tools provided by SAFE-FLOW are given in a number of supporting Technical Reports that are tabulated in this Chapter and referenced throughout this document.

1.2 Objectives

The main objective of the SAFE-FLOW JIP was:

To develop guidance, calculation methods and risk assessment procedures for green water and wave impact loading ('water impact loading') appropriate to design at each stage of a floater project, namely: concept development or conversion specification, detailed design and operational assessment.

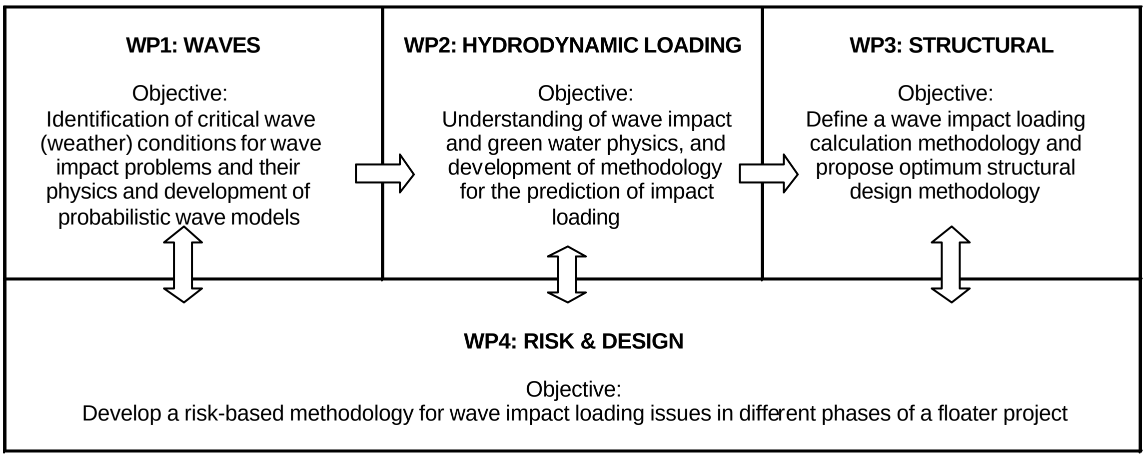
The main deliverables of the project are methodologies and design tools for the evaluation of water impact loading in the design of FPSOs (and drill ships) and design guidance based on the application of these tools and interpretation of project data.
The project was implemented in such a way as to make the results readily available to designers, oil companies, shipyards, ship operators and classification societies.

1.3 Organisation

The SAFE-FLOW JIP focused on four main tasks or work packages important to the solution of the problems of green water and wave impact loading:

- WORK PACKAGE (WP) 1 - WAVES: Identification of critical wave (weather) conditions for wave impact problems and their physics; development of probabilistic wave models, comparison of model test wave conditions with offshore measurements. Responsibility: Instituto Superior Technico (IST)
- WORK PACKAGE (WP) 2 - HYDRODYNAMIC LOADING: Understanding of wave impact and green water physics; development of methodology for the prediction of impact loading; model test data to support load calculation procedures. Responsibility: MARIN, University of Groningen, FORCE
- WORK PACKAGE (WP) 3 - STRUCTURAL: Define structural models appropriate to the assessment of green water loading and a wave impact loading and response calculation methodology. Responsibility: Atkins, University of Glasgow and Strathclyde, BP
- WORK PACKAGE (WP) 4 - RISK AND DESIGN: Develop a risk-based methodology for wave impact loading issues in different phases of a floater project. Responsibility: PAFA Consulting Engineers, Bureau Veritas, IZAR FENE

The important relationship between these work packages and a summary of their main objectives is illustrated in the following diagram.



WP 2 contained two distinct, major contributions in the fields of:

- Model testing, physical interpretation of hydrodynamic loading data and software development.
- Development of computational fluid dynamic modelling for simulation of impact loading in realistic scenarios.

The overall project management was carried out by MARIN.

1.4 Execution

The SAFE-FLOW project began in January 2001 and was completed over a three year period from 2001 to December 2004. The final reports were issued and revised over the early months of 2004.

The project was performed into two phases:

- The first phase, finishing in the final quarter of 2001, was aimed at evaluating the problem in a qualitative manner and gathering data that could be used to plan the second phase in more detail.
- The second phase, ending in December 2003, required presentation of analysed quantitative data; development of design and reliability methodology together with tools for tackling these methods and formulation of design guidance in the context of risk assessment appropriate to different phases of a typical FPSO project.

BP made available to the project full-scale measurements on the bow of the Schiehallion FPSO. A report has been issued covering measurements made over the period January 2000 to January 2003 as one of the SAFE-FLOW deliverables. These measurements have been important in confirming the orders of magnitudes of wave slam design loads and their probabilities of occurrence and in constraining the interpretation of model tests when it became clear that some model test results would prove difficult to interpret at full-scale.

Model tests were run in the wave basins at MARIN during the first two years. Numerical results were also developed over the whole 3 year period. For the first 2 years, these were based on existing publications, on previous data gathered at MARIN within the JIP 'F(P)SO Green Water Loading' and on phase 1 data from SAFE-FLOW. Over the duration of the SAFE-FLOW project, parallel model tests and analyses were being performed under a separate research project into bow slam at Glasgow University funded by the UK Engineering and Physical Sciences Research Council (EPSRC). This research included model tank tests on floating models of the BP vessels, Loch Rannoch (a shuttle tanker), and the Schiehallion FPSO to measure the characteristic effects of bow slams, forefoot slamming and hull girder bending.

The interaction between this work and the SAFE-FLOW project was very useful for the joint understanding of the related physics. The EPSRC research is reported openly elsewhere, see Barltrop and Xu (2004).
The final year of SAFE-FLOW has been used to review all results, develop design and reliability assessment methodologies based on the findings and present conclusions in the context of design for FPSOs against a background of risk assessment.

1.5 Organisation of project deliverables and associated documents

The present summary report is written to allow an easy access to the results of the SAFE-FLOW project and to summarise the design guidance and assessment methodologies developed. The structure of this report with its chapters is given in the table below:

SUMMARY REPORT On Design Guidance and Assessment Methodologies for Wave Slam and Green Water Impact Loading						
Introduction to the SAFE-FLOW project	History of FPSOs and water impact loading	Design methodology and limit states	Green water	Wave impact	Water entry	ComFLOW development

It is supported by a number of more detailed technical reports. An overview of these technical reports is given below:

SUPPORTING TECHNICAL REPORTS On Design Guidance and Assessment Methodologies for Wave Slam and Green Water Impact Loading
WP1 Technical Reports "Probalistic models of Wave Parameters for the Assessment of the Probability of Green Water" and "Models of Wave Steepness to Assess the Probability of Wave Impact in Bow Structures" (IST)
WP2 Experimental Technical Reports "Prediction of Wave Impact Loading" WP2 data reports: "Green water loading" "Loads on External Turret" "Bow Slam Loads on Schiehallion FPSO" "Validation Tests" *Program BowLab, Version 1.0* (MARIN)
WP3 Technical Reports "BP Schiehallion Full-Scale Monitoring – January 2000 to January 2003" "Structural Design Technical Report" (Atkins)
WP4 Technical Reports "Green Water and Wave Slam: Long-Term Analyses, Reliability Methods and Development of Design Guidance" (P A F A Consulting Engineers) "Class Rules Criteria for Green-waters and Wave Impact Loading – A Review of present Rules for Green-waters and impact loads and recommendations for improved Rules" (Bureau Veritas)

Beside the development of design guidance and assessment methodologies, another important aspect of the SAFE-FLOW project was the development and benchmarking of the ComFLOW program for water impact loading.

The technical reports related to this development are shown below:

TECHNICAL REPORTS COMFLOW DEVELOPMENT
WP2 Numerical Technical Reports "ComFLOW development 2nd phase" "Manual ComFLOW V1.0.5E and V1.1.5" *Program ComFLOW versions V1.0.5E and V1.1.5* (RuG, University of Groningen) "Benchmarks of the ComFLOW Program" "COMDIFF: Specification of Diffraction Interface" (FORCE Technology Norway)

For completeness sake, the table below gives an overview of reports delivered at the end of phase 1. Most of them are developed further into the phase 2 reporting.

Report name	Issued by	Issue date
Report of Workpackage 1 - Waves	I.S.T.	1 Nov. 2001
Annex of Report of Workpackage 1 - Waves	I.S.T.	1 Nov. 2001
Discussion Problem Identification	MARIN	27 Sept. 2001
Progress Report Numerical Tool Development	MARIN RuG CorrOcean	1 Nov. 2001
Report on Bow Monitoring Results from BP Schiehallion	WS Atkins	31 Oct. 2001
Green water and Slam Loading Structural Analysis Methodology Ph.1 Report	WS Atkins	Oct. 2001
Green water and Slam Loading Structural Analysis Methodology Ph.1 - Appendices	WS Atkins	Oct. 2001
Preliminary Design Guidance for Wave Slam and Green Water Events	PAFA Cons.Eng.	Nov. 2001
Outline Risk Assessment Model	PAFA Cons.Eng.	26 Oct. 2001

1.6 Glossary of terms

The following terms and abbreviations are used throughout this document:

ALS **Accidental Limit State** – The Limit State that requires the structure to be designed to remain essentially intact (but accepting some limited damage, yield, deformation, buckling, etc.) in response to accidental events or rare extreme loads (occurring, typically, with an annual probability of 10^{-4}). ALS conditions also require that the structure remains operable after damage to allow evacuation/repair in reduced subsequent environmental events.

ASD **Allowable Stress Design** – A traditional (increasingly less common) design approach in which permissible stress levels are defined that should not be exceeded by applied design load effects.

DAF **Dynamic Amplification Factor** – The ratio between the full dynamic response (typically displacement) of a structure to a given load time history and the simple static elastic response, based wholly on the stiffness of the structure. Dynamic amplification can be greater than or less than unity, the latter if the loading occurs too rapidly for the structure to fully respond.

EMLF **Elastic Mean Load Factor** – For ALS design, the factor, derived by structural reliability calculation, to be applied to the mean loading (the mean pressure obtained by experiment) to give the Equivalent Elastic Design Load (the level of load to be adopted in structural design, using simple elastic design principles).

EDL **Equivalent Design Load** – The level of uniform load (typically pressure) that is representative (gives the same structural design) as the true temporal and spatial variation of load. For ALS design, the EDL may be elastic of inelastic.

EEDL **Equivalent Elastic Design Load** – For ALS conditions, the level of load (typically pressure) such that design of the structure, if performed statically and elastically (so as not to exceed yield), will produce the same structural size as would be required to satisfy all limit states, including the ability to resist extreme (ALS) loads, albeit with some damage.

EIDL **Equivalent Inelastic Design Load** – For ALS conditions, the static level of load (typically pressure) on structural components that is equivalent to the full temporal and spatial variation of load that results from design water impact conditions, and for which the structure may be designed inelastically. Since the structure may be designed inelastically to this load, the EIDL is typically greater than the EEDL.

FEA **Finite Element Analysis** – The process of analysing a structure by splitting it into relatively regular elements of known behaviour in a mesh pattern so that the behaviour of the structure as a whole to load and vibration can be calculated by numerical methods on the computer.

FLS **Fatigue Limit State** – The Limit State whereby failure by repeated load cycling, resultant crack growth and subsequent fracture is avoided by checking that the structure has suitably fatigue durability (a design life in excess of the Target Life, given the expected load history). The Target Life is typically 3 times the design life of the structure for inspected locations, and 10 times for non-inspected locations.

FPRBD	**First Principles Reliability Based Design** – A design approach where sizing of the structure is performed to directly achieve a probability of failure lower than an allowed maximum, or where mean load factors are defined for a particular load condition and type of structure so that structural design using these factors will achieve this aim.
FPSO	**Floating Production, Storage and Offloading system**
FSU	**Floating Storage Unit**
GoM	**Gulf of Mexico**
IMLF	**Inelastic Mean Load Factor** – For ALS design, the factor, derived by structural reliability calculation, to be applied to the mean loading (the mean pressure obtained by experiment) to give the Equivalent Inelastic Design Load (the level of load to be adopted in structural design using inelastic methods). The IMLF is typically greater than the EMLF.
LPSF	**Load Partial Safety Factor** – In Limit State Design, the factor to be applied to increase each type of loading on a structural component to maintain a margin of safety in structural design.
LRFD	**Load and Resistance Factor Design** – A design approach whereby structural resistance reduced by material partial safety factors is required to exceed applied load effects that are increased by load partial safety factors, both sets of factors being defined to produce a suitably low probability of failure.
LSD	**Limit State Design** – A design approach whereby the structure is required to satisfy a number of requirements, such as strength (ultimate limit state), serviceability (serviceability limit state), fatigue durability (fatigue limit state) and resistance to accidents and rare severe loads (accidental limit state).
MLF	**Mean Load Factor** - The factor, derived by structural reliability calculation, to be applied to the mean loading (the mean pressure obtained by experiment) to give the Equivalent Design Load that will produce the required probability of failure
MPSF	**Material Partial Safety Factor** – In Limit State Design, the factor to be applied to reduce the material strength or resistance of a structural component to maintain a margin of safety in structural design.
RBD	**Rule Based Design** – A design method where structural sizes are prescribed by relatively simple Classification Society rules based typically on empirically calculated uniform load levels and simple structural response.
SLS	**Serviceability Limit State** – The requirement that a structure should be serviceable (minimal deflections, vibrations and no yielding) when subjected to specified (normally annual) loads with load and material partial safety factors of 1.0.
ULS	**Ultimate Limit State** – The design condition (limit state) that all components of a structure (and the structure as a whole) should have adequate strength (when reduced by appropriate material partial safety factors) to resist design loads of a specified magnitude (normally an annual probability of 10^{-2})
USR	**Ultimate Strength Ratio** – The ratio by which the ultimate capacity of a structural component (the load at failure or the load at which strain limits are reached) exceeds its elastic design capacity (the load at first yield).
WSD	**Working Stress Design** – Another term for allowable stress design

2 HISTORY OF FPSOS AND WATER IMPACT LOADING

2.1 FPSOs

Throughout the following text, unless explicitly stated otherwise, the acronym FPSO is taken to include both ‘floating production, storage and off-loading systems’ and ‘floating storage units’. The majority of findings will also apply to drill ships.

Although the hulls of most FPSOs are similar to those of trading tankers (frequently they are converted from tanker hulls) they have to operate in very different conditions. Notably, most FPSOs must remain on station during all weather conditions, although some may disconnect from their moorings (often in areas affected by tropical revolving storms). Operational procedures may vary to cope with extreme storms but in most cases the vessel structure must withstand any extreme loading conditions that can arise on site over the life-time of the project.

FPSOs are usually held in position by a mooring system. For harsh NW Europe, Gulf of Mexico (GoM) or Brazilian conditions, the interface between the hull, mooring system, production well-flow and export risers will normally be a turret. In most severe-weather locations, the turret will be housed internally with some type of chain table supporting the moorings below the keel. In more benign locations, the turret and chain table may be supported externally, above mean water level on a structural arrangement protruding from the bow or stern. Where metocean conditions are more uni-directional and not too severe, West of Africa for example, a spread mooring may be used that restricts yawing movements of the FPSO.

A turret also allows an FPSO to rotate relative to the prevailing weather such that, in a storm, the global environmental loads and roll response are reduced by maintaining the bow towards the main direction of the in-coming waves. This exposes the bow to high, local, water impact loads but, depending on the angular spread of forces and the yaw response, protects other parts of the hull from such loadings to a greater or lesser extent.

The occurrence and magnitude of water impact loads on the keel, hull, turret or topside of an FPSO will also be strongly dependent on the load condition (draft) of the FPSO. This will be affected by procedures adopted for day-to-day vessel operations. Since SAFE-FLOW did not set out to establish data on these two aspects of vessel operations, somewhat simplified models of long-term heading and draft variation are adopted within this and other SAFE-FLOW reports. Usually, these simplified models will need to be replaced by more precise models in order to apply SAFE-FLOW results to any particular FPSO.

2.2 Water impact loads: green water, wave slam and water-entry slam loads

2.2.1 Green water

Green water events arise when the hull is overtopped by a substantial quantity of water that subsequently flows across the deck. Green water can be a danger to operating crews and the resulting water impact loads may cause damage to topside structure, equipment and pipe or cable supports. FPSOs have been found to be vulnerable to such incidents.

Green water over the bow, sides and stern of typical FPSO structures is considered in this document. The work specifically considered the following design problems:

- Depth of water and speed of flow across deck following a freeboard exceedance.
- Impact pressures on deck and deck-mounted structures due to freeboard exceedance deck.
- Forces or impulses on large and small deck structures (due to green water).

The range of structural components likely to be subject to green water loading includes the following:

- Protection structures (bulwarks, breakwaters, etc.).
- Major items of deck equipment (significantly deflecting or diverting flow).
- Minor items of deck equipment (not significantly affecting flow).

2.2.2 Wave impact loads

High water-impact loads can also occur when a rapidly advancing, steep wave encounters the hull of an FPSO. This type of loading mechanism is usually referred to as wave slam, although where fine distinction is required it may be more precisely referred to as wave slap. These loads occur most commonly in stormy weather when the bow of the vessel is pointing towards the incoming waves. These are the events investigated in SAFE-FLOW so the alternative expressions bow slam or bow slap may be used occasionally.

The effects of wave impact on external surfaces of an FPSO have been evaluated during the SAFE-FLOW project. Measurements have been made of:

- Impact pressures for local design of the shell.
- Pressures on large areas of panel.
- Forces or impulses for large components of ship structure.

2.2.3 Water entry loads

In some conditions, the keel of a tanker can lift clear of the water in one wave to plunge rapidly into the water in the next. Water-entry events like this constitute another form of water impact or slam loading. Water entry loads on the keel or an internal turret tend to occur only when the FPSO is at a very low draft. However, an external turret can experience similar loads that are more likely to occur when the FPSO is at deeper draft.

Another water impact loading scenario that would appear to be a combination of the wave slam and water entry mechanisms occurs when a vessel's bow flare plunges into on-coming waves. In this instance, substantial relative velocities between the hull and water particles may be caused by the vessel's pitch response but high velocities of submergence may also arise due to the velocity of the wave profile.

2.2.4 Focus of the SAFE-FLOW project

Collectively, all the mechanisms just described are referred to as water impact loading. In the following, separate sections recommend how measurement data, design calculations and tools be used and the design guidance that has been drawn up by the SAFE-FLOW project for green water, wave slam and water-entry slam respectively. Turret slamming and bow flare slam are treated together as though primarily due to the water-entry.

The nature of bow impact load is that it is neither fully correlated in time nor in space. Therefore pressures for local design will tend to be higher than pressures for global response estimation. Furthermore, owing to the large differences in local and global force, rise times and structural natural periods, the most appropriate methods for handling the different types of structural response may vary. So, for some components a design pressure or force might be the most useful design value whereas for other components a design impulse might be most appropriate. Consequences of exceeding design strengths will also be considered: for example the permanent deformation in some bottom plating may be less onerous than overloading the hull girder.

2.3 Damage arising from water impact loading

There are a number of recorded instances of green water damage to FPSOs sometimes occurring soon after the vessel is installed:

- Green water damage on equipment on the bow of the Alba FSU (Floating Storage Unit) in a North Sea winter storm in January 1995. The unit was out of operation for a number of weeks.
- In the same storm damage occurred on the Emerald FSU, resulting in the complete destruction of a fire fighting platform, as shown in Figure 2-1. In addition to the evident direct damage, this incident also affected the safety of the FPSO against fire.
- Green water damage to fire equipment storage and a crane were registered on the Norne FPSO in 1998 (March 19) due to green water from the side of the vessel. Analysis revealed that the vessel was also vulnerable to damage from green water coming over the bow. This resulted in operational draft restrictions to minimise the chance of shipping green water.
- In January 2000 the living quarters on the bow of the Varg FPSO was hit by green water. This resulted in the damage of a window at the second floor and flooding of the area behind it, see Figure 2-1.

Figure 2-1: Damage to fire fighting platform of Emerald FSU (left) and the damage and first repair of a window after the accident on the Varg FPSO

Based on these incidents the Health and Safety Executive (HSE) in the UK performed in co-operation with the Norwegian Petroleum Directorate (NPD) a study into the vulnerability of all 16 UK sector and 5 Norwegian FPSOs, which is summarised in Figure 2-2. In this figure the susceptibility of green water is identified at three levels: low (green), medium (orange) and high (red).

FPSO Green Water Susceptibility

Vessel UKCS FPSOs	BOW	SIDE 15 deg	SIDE 30 deg	STERN
1	MED	MED	HIGH	LOW
2	HIGH	MED	HIGH	MED
3	HIGH	HIGH	HIGH	LOW
4	HIGH	MED	HIGH	MED
5	HIGH	MED	MED	LOW
6	LOW	MED	HIGH	MED
7	LOW	MED	HIGH	MED
8	HIGH	MED	HIGH	LOW
9	HIGH	HIGH	HIGH	MED
10	MED	MED	HIGH	MED
11	MED	MED	HIGH	MED
12	MED	MED	HIGH	MED
13	LOW	LOW	MED	LOW
14	MED	MED	HIGH	MED
15	LOW	LOW	MED	LOW
16	MED	MED	HIGH	MED
Norwegian FPSOs	BOW	SIDE		STERN
17	HIGH	HIGH	?	LOW
18	LOW	HIGH	?	LOW
19	HIGH	MED	?	?
20	MED	MED	?	?
21	MED	MED	?	?
Low risk generally < 3 m freeboard exceedence				LOW
Medium risk generally 3 to 6 m freeboard exceedence				MED
High risk generally > 6 m freeboard exceedence				HIGH

Figure 2-2: Susceptibility of green water of North Sea FPSOs

All FPSOs are identified as having a medium (29%) to high risk (71%) of shipping critical amounts of green water on board from the side of the vessel (wave heading 15 and 30 degrees from head waves), summarising:

	High risk	Medium risk	Low risk
On bow	38%	38%	24%
From side	71%	29%	0%
On stern	0%	56%	44%

Wave impact damage has been experienced by both the Foinaven and Schiehallion FPSOs. During the night of the 9th November 1998, in a sea state estimated as H_s = 14 m, T_p = 15-16 seconds, an area of forecastle plating on Schiehallion above the main deck, between 15 and 20 m above notional mean water level was pushed in by 0.25 m.

There was some associated minor plating deformation inside the fore peak, below the main deck but there was no damage to the flare supports (which are mounted on top of the forecastle) or any process equipment. The damage occurred at the time in the storm at which the measured wind gust speeds were strongest but at the time the wind sensors on the vessel recorded a 10-minute gust speed of 59 knots compared with a one-year-return-period design value of 69 knots. By contrast, the most severe vessel motion, due to heave and pitch, occurred between 2 and 6 hours later. Wave records from a vessel some 12km distant from Schiehallion showed a rapid increase in wave height in the period leading up to the damage event. A mean zero crossing period of 11 s, coupled with a significant wave height of 14 m indicates a severe sea state steepness estimated as 1/13, but there are no corresponding records of individual waves.

Figure 2-3: Photograph showing wave impact damage to bow of Schiehallion FPSO (from newspapers, taken by Greenpeace)

Wave impact damage to the bow of the Schiehallion, resulted in an evacuation of personnel, a significant deferral of oil production, expensive offshore hull repairs and an upgrade of the complete bow structure of this FPSO. The estimated direct cost of repairs was around US$5 millions.

2.4 Review of design and specification requirements

In a review on behalf of the United Kingdom's Health and Safety Executive (HSE), PAFA Consulting Engineers implemented a review of green water and wave slam design and specification requirements for FPSO/FSUs. The main conclusions of that review were that design against green water and wave slam was haphazard. In many cases there was little or no explicit recognition of the potential problems and hence no specific allowances in the design process. The SAFE-FLOW phase one report summarises some of the specific points made.

Where projects did recognise the potential dangers of green water incidents, model testing might be assumed to allow adequate investigation, but it was unclear whether model test programmes were sufficiently well specified to allow green water and wave slam events to be detected with certainty. Even when green water events were present in model test results, these could go unrecognised until problems were encountered after the vessel was in operation.

There was a tendency among designers to identify FPSO/FSUs as tankers and to assume that redundancy and conservatism inherent in the Classification Society Rules would be sufficient to protect structure against slam damage. However, the Classification Society Rules from that period (pre-1999) were non-conservative when applied to a vessel permanently moored in an exposed location. They did not recognise, adequately, the extreme wave slam loadings that would occasionally, if infrequently, be experienced by a static hull.

Many of the issues raised above have been taken up by the Class Societies and Rules have been updated to cope with the inherent dangers. Within SAFE-FLOW, Bureau Veritas has made a separate review to comment on the current adequacy of Class Society Rules or indeed to point out whether further work may be necessary (see Technical Reports WP 4).

2.5 General notes on risk and design against water impact loads

From the historical data presented above and many reports of other incidents, it is clear that water impact loads can generate large loads that have the potential to cause significant damage to an FPSO (or Floating Storage Unit). For this reason it is important to recognise water impact loading as a hazard to floating units and make appropriate allowances in design. This is especially the case given the likely high cost of making a repair to an operating vessel in the field. Operational experience to date confirms the capacity for these loading mechanisms to cause damage, but so far, there have been no reported fatalities, vessel losses or serious pollution incidents attributed to these causes. In this last respect, water impact loading joins a list of potential hazards that present risks to a floating vessel. Given the likely interaction between design for water impact and other essential vessel systems, it is essential to consider all potential hazards and develop a risk management strategy for the vessel as an integral part of the design, operation and maintenance procedures that need to be considered throughout design and the working life of the vessel.

The UK Health and Safety Executive recommend a general hierarchical approach to risk management. The earlier in the chain that each hazard can be eliminated, the more beneficial it is for overall safety. The hierarchy used by the UK HSE is as follows:

- Avoid - eliminate the hazard thereby removing the risk altogether.
- Minimise - make changes that reduce probability of occurrence and/or consequence of a hazard.
- Substitute - replace the activity with one which is less risky.
- Control - introduce procedures or hardware that allow actions to be taken to reduce the overall risk.
- Mitigate - by designing to reduce the effect of the identified hazard.

Some operators find it helpful to categorise risks according to whether or not they represent safety or business critical issues so that they can given higher priority as required. Generally, each of the water impact loads addressed within SAFE-FLOW has the capacity to be safety and business critical for most FPSO configurations that could be deployed in a severe environment (such as the North Sea). They may or may not be critical in moderate weather conditions depending on the type of vessel deployed. However, in all cases the risk needs properly assessed (and, if necessary, categorised accordingly).

The earlier in the design cycle that a critical risk is identified, the more effectively it can be managed. Hence it is important to make some assessment of relative importance of each water impact load mechanism either during concept design or during early considerations of a proposal to convert a vessel. This might be done by considering historical precedent or by using some of the tools provided in the SAFE-FLOW reports. However, it is unlikely that full details of a vessel configuration or operating procedures will be available early in design so it will generally be necessary to make some provision based on historical precedent and leave a detailed review until these have been defined.

2.6 Bibliography and references

In general literature, a lot of research is available on the subject of water impact loading. This literature has been used as part of the research in SAFE-FLOW. References are made to this literature at the appropriate places in this report and the supporting technical reports. A general (not complete) list of literature is given as Appendix to this report to support further research into this subject.

3 DESIGN METHODOLOGY AND LIMIT STATES

3.1 Requirements

It is recognised that water impact loads on FPSO structures are complex, comprising potentially impulsive loads that result from random events. Many different parameters have an impact on the magnitude of response to these loads, including wave height, period and shape, hull form, vessel motion, heading and draught, local freeboard, deck congestion and the geometry and location of the components being loaded. The response of these components, which may be of varied structural configuration, materials and quality of fabrication, may be dynamic and potentially inelastic (i.e., damage may be accepted at extreme load).

Any structural design methodology must therefore be based on a thorough understanding of:

- The characteristics (spatial and temporal variation) of the expected loading.
- The probability of occurrence of peak loads which will define the magnitude of 'design' events for the vessel geometry selected.
- The likely static/dynamic and permissible elastic/plastic response of components of the structure at this level of loading.
- The quality of materials, manufacture and welding and the effect of these on structural capacity.
- The consequence of failure and hence the extent of damage to the structure that is permitted for these extreme design events.
- The recurrence intervals of significant loads, which will impact on fatigue durability and maintenance requirements.
- The need to provide load paths to carry this loading back into the main structure.

In addition to reflecting all of the above characteristics of load, it is a stated aim of the SAFE-FLOW project to provide assessment and design methodologies for the following stages in the vessel design and maintenance processes:

- The conceptual and feasibility design of structures subject to extreme green water and wave impact loads.
- The subsequent detailed design of such structures.
- The design of novel or complex structures that are not covered directly by this guidance.
- The adaptation of existing designs to suit new environments or operations.
- The assessment of existing structures that may not have been designed on the basis of the guidance herein.
- The refined detailed design of structures to provide weight saving or to meet other design restraints.
- The assessment of previously damaged structures against the likelihood of subsequent 'events'.

3.2 Current structural design methodologies

3.2.1 General

The following subsections introduce and describe the four primary methods of structural assessment that are used in modern design rules and regulations:

- Rule Based Design, where structural sizes are defined directly using empirically or historically based equations and formulae from Classification Society Rules.
- Working or Allowable Stress Design, where structural sizes are defined such that calculated stresses in components are kept within specified allowable limits.
- Limit State Design, which defines a number of criteria (strength, serviceability, fatigue, etc.) that must be satisfied, using load and material partial safety factors to provide a required level of safety.
- First Principles Reliability Based Design, which requires that structural components be sized to achieve a given level of reliability (annual probability of failure less than a specified value, depending on failure consequences, redundancy, etc.).

These possible approaches to water impact design are discussed and reviewed in the following sections.

3.2.2 Rule Based Design (RBD)

Most ship and FPSO Classification Society Rules provide a prescriptive approach to structural steel design for bow impact and green water loading. This approach typically employs two stages:

- Determine the magnitude of design pressures from empirical equations based on expected maximum wave conditions or ship dimensions.
- Evaluate the required plate thickness or section size of the component being designed directly using this design pressure.

Design pressures are typically based on best available theoretical relationships, supplemented by experimental coefficients and experience of vessel behaviour in a variety of seas. Structural response calculations are generally relatively simplistic (linear elastic). The most significant advantage of this method is its ease of use. This makes the approach well suited to preliminary design, since structural component sizes can be obtained very quickly.

However, the primary concerns over such a methodology are as follows:

- The sources of data used and hence the limits of applicability of the design approaches generally are not specifically stated.
- The environment of operation of the unit is not taken into account explicitly.
- Typically, the true nature of the loading is not explained, with the result that important design issues (impulsive loading, dynamic response, resulting fracture, load paths into and through supporting structures, etc.) are not explicitly addressed.
- Existing material fabrication quality requirements may not be sufficient for plastic design for impulsive loading that characterises bow impact and green water events.

Nevertheless, rule based design methods remain a valuable tool provided that their limitations are known and they are supported by more appropriate assessment methods where these limitations are exceeded.

3.2.3 Working or Allowable Stress Design (WSD or ASD)

In Working Stress Design, allowable stresses are defined for different failure modes, such as axial tension, shear, buckling and combined stress. Structural members are then sized to ensure that stresses or combinations of stress in the structure are within these allowable limits. This approach has traditionally been used in onshore and offshore structural design for many years and has the advantage of being well understood and widely accepted in such fields.

The chief limitation of working stress design is that a single factor relates permissible stress to material yield. The same factor is incorporated irrespective of the origin or nature of the applied loading, albeit with an overload allowance for extreme environmental loads (the overload allowance is applied to dead load as well). This approach cannot be expected to reflect the different uncertainties and variabilities in loads that originate from different sources (dead load, vessel motion, impact load, etc.). Therefore the method will not, in general, provide uniform structural reliability over these different load types and combinations.

A further limitation is that the safety factors inherent in the allowable stresses used are largely historically based derived in the most part in response to failures of typically onshore structures. The level of safety imposed by these allowable stresses is therefore not necessarily appropriate to the probabilities of bow impact and green water loading, which are characterised by relatively few, very high intensity loads. There is little precedent for calibrating the allowable stresses against load types that differ significantly from the historical variations inherent in the rules.

For the above reasons, Working Stress Design methods are generally being replaced by Limit State Design codes, where individual factors are used to account for varying load and resistance effects.

3.2.4 Limit State Design (LSD)

Most modern structural codes address the design of structures via a series of 'Limit States'. The number and terminology of these varies from code to code, but the following is taken from the latest ISO standards:

- Ultimate Limit States (ULS) are used to define the required strength of a structure to safely resist expected loading. Uncertainties in the magnitude of this load are reflected in 'Load Partial Safety Factors' and uncertainties in structural response and material behaviour are accommodated by 'Material (or Resistance) Partial Safety Factors'. This approach is also referred to as Load and Resistance Factor Design (LRFD).
- Serviceability Limit States (SLS) are used to ensure that structural deflections, vibrations, etc. are not too extreme for the intended use of the structure.

- Fatigue Limit States (FLS) are used to ensure that the structure has sufficient durability to prevent premature failure by fatigue crack propagation.
- Accidental or Progressive Collapse Limit States (ALS or PLS) are used to ensure that the structure has sufficient strength to resist rare extreme or accidental loads, albeit with some damage. Other ALS limit states are also used to ensure that the resultant damaged structure is able to remain intact under subsequent expected load levels.

Load and material factors are typically set to unity for SLS, FLS and ALS checks. Suitable safety is provided by choosing an appropriate return period for SLS or ALS conditions, and by setting a target life in excess of the design life for FLS conditions. ULS load and material partial safety factors are in the region of 1.3 and 1.15, respectively, giving a combined safety factor of about 1.5.
Typical annual probabilities of exceedance for different limit states and different load types are given in the following table (from DNV rules):

Load Category	ULS	PLS(ALS)		SLS	FLS
		Intact	Damaged		
Permanent	Expected				
Live	Specified				
Deformational	Specified				
Environmental	10^{-2}	10^{-4}	10^{-1}	Specified	History
Accidental	n/a	10^{-4}	n/a	n/a	n/a

Limit State checks are increasingly used in structural design because of their thoroughness and their ability to be calibrated by assigning different load and material factors to accommodate the effect of different types of load and construction, so that a required level of structural reliability (maximum probability of failure) is achieved. Thus dead loads, for example, can be assigned lower load factors than wave loads, because of the greater certainty in their magnitude.
This calibration of load and resistance safety factors is permitted and required in regional annexes to suite the specific probabilities of load due to environmental effects in different parts of the world. Thus there is precedent for adopting a calibrated design approach to water impact loading within the Limit State Design method.

3.2.5 First Principles Reliability Based Design (FPRBD)

It is also possible to size structures directly to achieve a given level of structural reliability or probability of failure. This approach is uncommon in design, because of the need to know not only the magnitude, but also the variability of different load and resistance components. The method is used primarily in the calibration of Limit State Design load and resistance factors for different types and geographical locations of structures. Reliability Based Design is useful where there is no prior history of load factors for various types of loading, or of material factors for certain types of construction. This is very much the case for green water and bow impact loads, whose variability and probability of exceedance is not necessarily the same as other forms of loading on a floating structure.

3.3 Selection of design methodologies

3.3.1 General

Within the SAFE-FLOW project, it is required to provide rules that cover all stages of structural design, from conceptual sizing, through detailed design to the reassessment of existing structures. Based on the above review of available structural design approaches, three separate structural methodologies will be considered for green water and wave impact loads, giving the widest possible flexibility to the designer. These methodologies are as follows:

1. A *first principles reliability based approach*, assessing the probability with which green water and slam loads will occur and sizing the structure to achieve an acceptable low probability of failure, determined according to the consequences of such failure. This approach would typically be used to determine Mean Load Factors for limit state design, or to assess the reliability of existing structures that may not have been fully designed against green water or wave impact. It is conceivable, but unlikely, that it could be used directly in design in proposing a member size to achieve the target reliability.
2. A *limit state approach*, using typical Mean Load Factors derived by the above reliability based method to provide a relatively simple but robust methodology for the detailed structural design or reassessment of existing structures.
3. A *first estimation approach*, providing an even simpler methodology that would be appropriate for preliminary, conceptual or even conservative detailed design. This methodology will be based on equivalent static design loads, derived to provide the same response as the more complex spatial and temporal variation of impact pressure. This approach has a parallel with Rule Base Design based on Classification Society rules in the sense that relatively simple formulae are used.

Working stress design methods will not be considered further, since these are effectively a less rigorous alternative to the limit state approach. The above approaches are described in more detail in the following sections. The reader should refer to Chapters 5 through 8 for the application of these methods in design against green water, wave impact and water entry loads.

3.3.2 Method 1 - First principles reliability based methods

Reliability based methods are characterised by the flowchart shown in the Figure 3-1. This illustrates in general terms how the approach may be used to derive Mean Load Factors for structural design, or optionally to actually perform this structural design.
The following actions are proposed:

- Vessel data (hull geometry, drafts and corresponding freeboards and RAOs), environmental conditions (scatter diagrams, headings) and target structure data (bow structure, deck structure and external turret) provide input to the method.
- Load effects (impact pressures, green water loads) are calculated from data derived during the SAFE-FLOW project, including probabilities derived from experimental scatter.

- Load probability data is coupled with uncertainty data associated with structural response (dynamic amplification, yield, stability, etc.) in a reliability assessment.
- Structural design is then optionally undertaken using the derived Mean Load Factors, or an existing structure may be assessed using the derived probability of failure for a particular member size. Alternatively, and where possible, the structure may be resized until a required probability of failure is achieved.

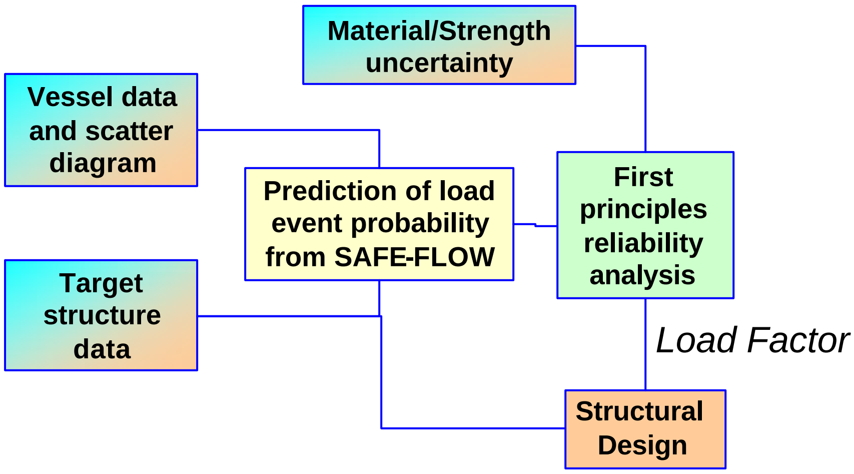


Figure 3-1: First Principles Design Approach Flowchart

The major advantages of the reliability approach are as follows:

- Allows load levels to be tailored to specific vessel and environmental conditions.
- Can consider complex combined probabilities of different drafts, directions, etc.
- Ideal to determine load factors for conditions not covered by SAFE-FLOW.
- Allows investigation of uncertainties in input data.
- Ideal for assessment of existing structures, without being too onerous.

The most significant disadvantages are:

- Not only is mean data required, but also variability, which is often not readily available.
- Unwieldy as a design tool, particularly in early stages of design.

Further description of the reliability method is given in Section 3.4, below.

3.3.3 Method 2 - Limit state design

A flowchart showing the path through a typical limit state design is shown in Figure 3-2, which illustrates how the approach may be used in combination with Mean Load Factors for structural design.
The following actions are proposed for limit state design:

- Compile vessel data (hull geometry, drafts and corresponding freeboards and RAOs), environmental conditions (headings and critical sea states selected as recommended herein) and target structure data (bow structure, deck structure, external turret).

- Determine most probable maximum load effects (impact pressures, green water loads) from data derived in the SAFE-FLOW project.
- Assign load factors to these pressures to produce design loads. Load factors have been derived as part of the SAFE-FLOW project and may be applied to similar structures and load conditions to those used within the project. For all other cases, load factors should be derived using the reliability approach.
- Structural design is then undertaken based either on factored time histories of load, or using equivalent design loads.

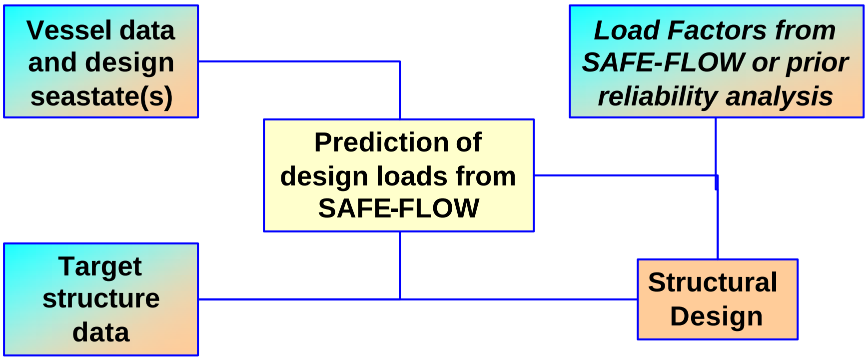


Figure 3-2: Limit State Design Flowchart

The following advantages are apparent with this method:

- Does not need as much complex data as the first principles reliability approach.
- Generally much quicker to perform than reliability based calculation due to limited sea states considered and the reduced data requirements.
- Can be tailored to a specific environment vessel and type of structure.
- Allows investigation of response in different sea states.
- Is well suited to the detailed structural design stage.

There are some disadvantages, however, as listed below:

- Relies on load factors derived from prior reliability calculations or from SAFE-FLOW.
- Selection of critical sea states is very important.
- Is still not very suitable as a concept design tool.

Further description of the limit state method is given in Section 3.5, below.

3.3.4 Method 3 - First estimation approach

The first estimation approach is provided within SAFE-FLOW for bow impact design, turret loads and bow flare impacts.
The basic approach is characterised by the flowchart shown in Figure 3-3. This illustrates in general terms how the approach may be used to derive required structural sizes. The following actions are proposed:

- Vessel data (bow geometry), structure data (size of structural target) and design wave data (maximum sea state) is obtained.
- Design pressures are derived from equations given by the SAFE-FLOW project.

- Structural design based on simple elastic design principles is then undertaken.

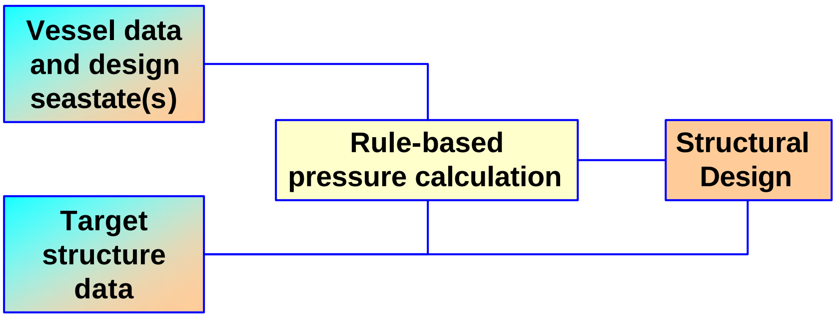


Figure 3-3: First estimation approach flowchart

Advantages of this method are as follows:

- Allows rapid calculation of required structural sizes.
- Inherently includes uncertainties and probabilities from SAFE-FLOW.
- Design pressure values can be directly compared with values in current design rules.
- Very suitable for preliminary or conceptual design or FEED.

Key disadvantages are listed below:

- Limited to structures and environments similar to those assessed in SAFE-FLOW.
- Selection of critical sea states very important.
- Probably needs to be conservative, since worst case must be assumed.

Further description of the first estimation method is given in Chapters 7 and 8, for wave impact and water entry cases, respectively.

3.4 Reliability based design for water impact loads

The objective of structural design is to minimise the risk of failure of components through the lifetime of the facility. In conventional limit state or working stress design methods, this is achieved indirectly by keeping stresses in the structure below predetermined levels. Such stress levels have been set by code developers, by calculation or by experience, to ensure a low enough probability of failure despite expected variability of loads and resistance.

Reliability methods, on the other hand, aim to evaluate the risk of failure of components directly. To achieve this, it is necessary to know the probability density of load, p(L), and of resistance p(R). From these, the probability of failure may be determined as the probability that the load is greater than the resistance, p(L>R). This is illustrated by Figure 3-4:

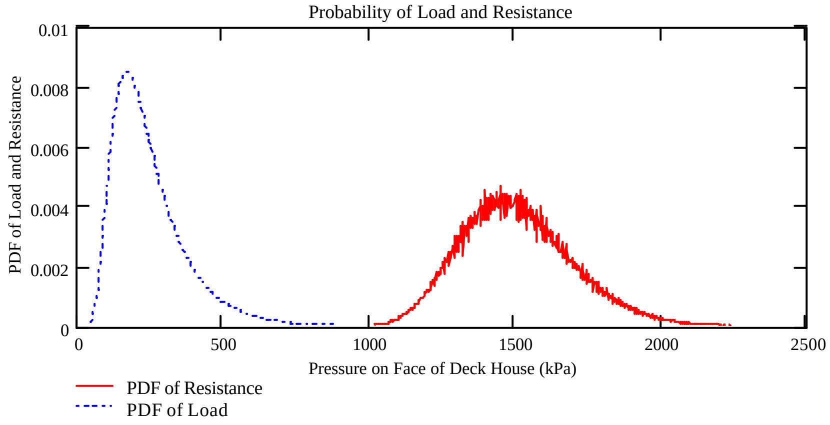


Figure 3-4: Example of Probability Density Functions (PDFs) of Load and Resistance

Within SAFE-FLOW, the probability of load is derived from mathematical or numerical predictions of freeboard exceedance or water surface vertical velocity, coupled with experimental results giving the design pressures and the variability of these pressures over the face of the target structure. Mean and variability of resistance is based on conventional bias and uncertainty in material and structural parameters, such as yield, coupled with uncertainty in dynamic response to applied loads.

First Principles Reliability Based methods may be used as follows:

- To assess the probability of failure for existing structures.
- To derive load factors for use in limit state design.
- To resize structures to achieve a required probability of failure.

Probabilities of failure are derived directly from the method, based on the overlap of the tails of the load and resistance probabilities, p(L>R). Such annual probabilities of failure should be compared with required values that are commensurate with a required consequence of failure, such as those given by DNV Classification Note 30.6 (Reliability of Marine Structures) in the following table:

Class of Failure	Consequences of Failure	
	Less Serious	Serious
Redundant Structure	$P_F = 10^{-3}$	$P_F = 10^{-4}$
Warning of Failure	$P_F = 10^{-4}$	$P_F = 10^{-5}$
Sudden Failure	$P_F = 10^{-5}$	$P_F = 10^{-6}$

In the case of FPSO structures, warning of failure may be assumed (in that access to areas that are at risk will be prohibited in severe seas), but structures are not normally that redundant. Damage to deck houses would be classed as serious failure, but damage to bow plating (unless it threatened the vessel) would be less serious. Hence annual probabilities of failure in the range 10^{-3} to 10^{-5} would normally be considered.

In the case where mean load factors are to be derived, the following approach has been adopted in SAFE-FLOW:

- A reliability calculation is performed and the is structure resized until a required annual probability of failure is achieved.
- A further calculation is performed based on a 'Design Load', being a design force or pressure with a certain probability of occurrence (e.g., a 1 in 100 year most probable maximum event).
- The Mean Load Factor is derived as the factor required to give the same structural size, from the Design Load approach, as from the reliability based approach.
- Thus the mean load factor, *if applied to the Design Load*, will result in a structure with the required probability of failure.

The text in italics above is very important. It is necessary to ensure that load factors are applied to the same probability of load for which they are derived. For example, load factors calculated on the basis of 100 year return events, should be applied only to 100 year design loads or pressures.

It is recommended that greater that 1 in 100-year conditions be considered as the basis of design because these produce more stable load factors and maximise the area of the structure that is affected by the loading to incorporate all structure that can be impacted by rare extreme loads. One in 10,000-year conditions are recommended to be consistent with ALS design criteria. Use of such severe events will not result in more severe checks on loaded areas (this is goverened by the selected probability of failure), but will affect the extent of structure subject to impact loading. This is further discussed in Section 3.5, below.

As a further enhancement to the method, elastic and inelastic mean load factors may be derived for design by elastic and plastic methods. This is further described in Section 3.7, below.
The third approach described above is to design the structure directly using the reliability approach. This is possible, but not common, for the following reasons:

- a significant amount of data is required by the approach, which is often not available in the design phase;
- the methodology is time consuming and not well suited to the pressures of the design process;
- by necessity, structural models in the reliability method are often simplistic, suitable for comparative purposes to derive mean load factors, but not well suited to detailed design.

Consideration should be given to the above before using the reliability method directly in design.

3.5 Limit state design for water impact loads

3.5.1 Conventional design considerations

It is normally required to satisfy all limit states, ultimate, serviceability, fatigue and accidental (ULS, SLS, FLS and ALS). Fatigue limit states are very different from the others and will be considered separately in this methodology. For the limit states that deal with structural strength, in certain circumstances it is clear from the probability and relative magnitude of the loads that particular limit states will not govern. In such cases, design for these limit states may be avoided ‘by inspection’.
For primary vessel design such as hull girder bending, SLS and rare environmental load

ALS events are usually not considered for the following reasons:

- Serviceability usually involves the structure remaining elastic and displacements/vibrations being within acceptable criteria for frequently recurring load levels. The normal requirement is to consider annual events with unit load and material partial safety factors. However, the relative magnitude of ULS loading (100 year return, load partial safety factor of 1.3 and material partial safety factor of 1.15) implies that SLS conditions will usually never govern strength requirements. For this reason, SLS criteria are seldom checked, only where significant deflections and vibrations are expected to limit operations on the vessel.
- For accidental limit states, unit load and material partial safety factors are again specified by the codes. It is true that the annual frequency of such events is less than for ULS, but ALS events will again not govern if the ratio between the 10^{-4} rare event loading and the 10^{-2} ULS loading is less that the cumulative partial safety factor for ULS conditions (approximately $1.3 \times 1.15 = 1.50$). This is illustrated by Figure 3-5 for typical design events that vary linearly with wave height (say hull girder bending).

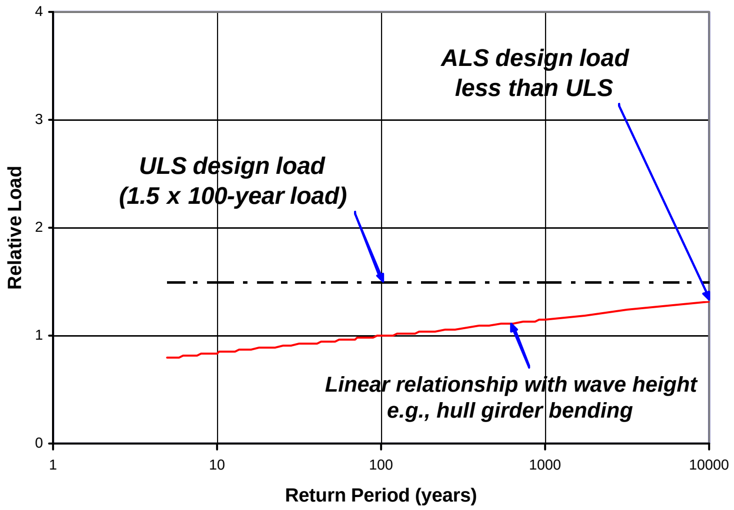


Figure 3-5: Comparison of ULS and ALS Design Loads - Load µ Height

- The normal approach to structural design against wind and wave loads is therefore to design in accordance with the ULS conditions alone:
- Define the loading that has a reasonable chance of occurring during the lifetime of the structure (typically 50 or 100-year storm events).
- Increase this loading by one or more partial safety factors, these safety factors being set to allow for uncertainties in load and resistance, to provide the required level of structural reliability and to safeguard against the possibility of more extreme unexpected events.
- Design the structure elastically at this factored load level, with an additional partial safety factor to reduce permissible material strength.

For water impact water events, however, the magnitude of rare environmental loading on the structure is not necessarily less than the factored ULS loading, and it may not be possible to ignore extreme loads in this way. This is a very important consideration and design for water impact events is discussed in more detail in the following section.

3.5.2 Extreme event design requirements

The probability of occurrence of water impact loads through the life of a floating structure is dissimilar from other forms of load, such as hull girder bending. Whereas the latter will steadily increase with the increasing amplitude of less frequent waves, the same is not true of green water and wave impact loading. For these types of loading, rare extreme loads can be generated that are not necessarily a simple linear extrapolation of less frequent loads: In particular:

- Green water may not occur at all during more frequent design events (the freeboard will not be exceeded by these lower crest heights), but green water will become significant under more extreme conditions (when the freeboard is exceeded).
- When green water loading does occur, structural response is generally proportional to a power (2 or 3) of the freeboard exceedance, resulting in very much more substantial loads from rarer, more severe events.
- Wave impact loads appear to be highly non-linear in that rare extreme events (extensive, simultaneous slams) produce significantly greater loading over larger areas than more conventional design events.

However, there is also a tendency for wave impact loads to be self limiting, in that extremely steep waves will tend to trap a cushion of air against the bow or even break before impacting the structure, in both cases reducing the magnitude (but possibly increasing the loaded area) of these pressure events.
With the exception of this self limiting effect, the result of the above will be to significantly increase the magnitude of extreme design loading for water impact (particularly green water) events, relative to the loading that would be used for ULS design.

This is shown by the example in Figure 3-6, assuming the ALS load to be proportional to the square, cubed and fourth power of the wave height associated with the return period:

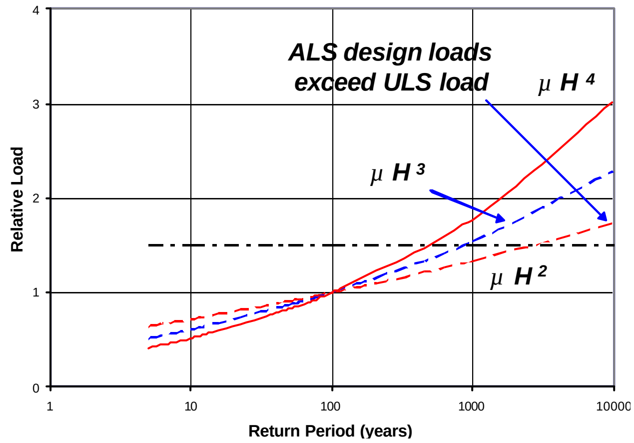


Figure 3-6: Comparison of ULS and ALS Design Loads for Wave Impact

In most of the above cases, the ALS condition will govern. This is not necessarily the case when the ULS limit is marginally exceeded, as design to ALS permits damage to the structure and is therefore not as onerous as the simple linear elastic requirement of the ULS criterion.

The situation can be even more extreme in the case where there is a clear threshold in the loading process, such as the exceedance of the freeboard in the event of green water. In the following Figure 3-7, the freeboard is assumed to be exceeded at a wave height with a return period of about a year and linear, squared and cubic relationships are considered between freeboard exceedance and loading.

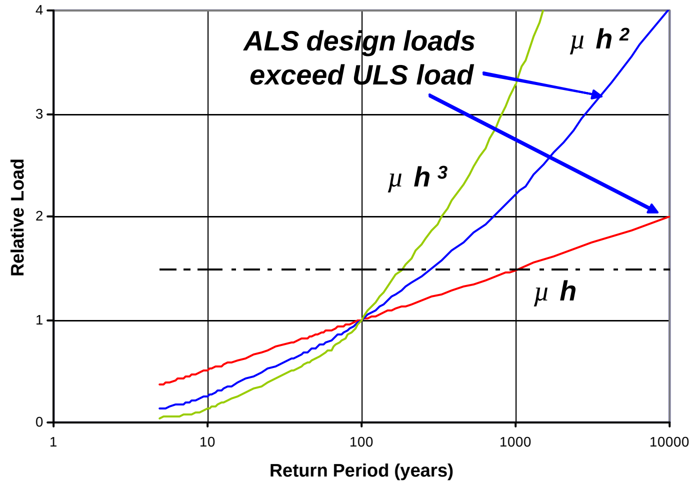


Figure 3-7: Comparison of ULS and ALS Design Loads for Green water

In this case, very much greater extreme loads can result from the rare wave occurrences with long return periods, such that ALS design will almost certainly govern over ULS design.

In the extreme (see Figure 3-8, below), the ULS loading for a certain return period can be zero, if the 100 year return relative motion does not exceed the freeboard. For longer return periods, the loading is finite and rapidly becomes significant. Such loading processes make the investigation of ALS conditions a necessity. It should be noted that the same effect will apply for structures high in the bow under wave impact loading.

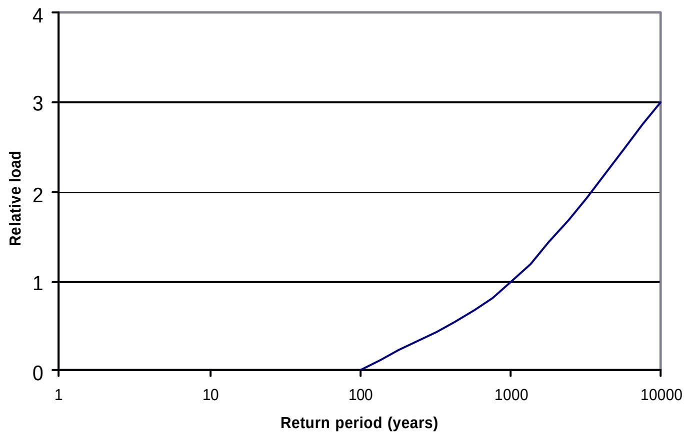


Figure 3-8: Green water loading with freeboard not exceeded for 100 year event

It is recommended that extreme events be used as the basis of the calculation of mean load factors (Section 3.4), since less severe events will produce poorly defined factors due to the cut-off in load below a certain design height. In the following Figure 3-9, the safety factor becomes infinite (no load to factor) if the design load is based on less than a 5-year return period sea state. The factor is more stable for longer return periods.

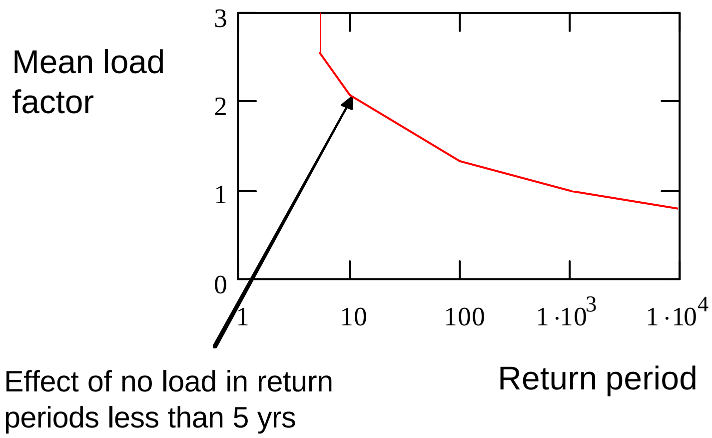


Figure 3-9: Mean load factor as function of return period

Note that in this case, the factor to apply to a 10,000-year condition is less than 1.0, indicating that this design event would produce a probability of failure less than the required value. Hence the 10,000-year loading may be reduced yet will still achieve the required probability of failure.

3.5.3 Proposed methodology

With the typical load probabilities described in the previous sections, the primary objectives of structural design for water impact problems should be as follows:

- The structure should have adequate margin of safety against damage due to more common events, such that there is no significant risk to safety or the environment, and so that maintenance costs are not excessive.
- The structure should have adequate reserve capacity, consistent with the consequences of potential damage, to resist more extreme loads in a ductile fashion (but without catastrophic or progressive failure).

It is likely that an approach considering only the ULS condition will not be adequate for wave impact and in particular green water loads, since the magnitude of extreme loads may be significantly beyond what may be inherently included via load factors and reserve strength beyond the ULS loading. In extreme green water loading conditions, ULS loads may be zero, yet deck structures may still be subject to rare extreme loads if 100 year wave conditions are exceeded.

The proposed approach in SAFE-FLOW is therefore to consider both rare extreme events with a low probability of occurrence (thus satisfying the ALS) in addition to lower design conditions and associated partial safety factors (for the ULS).
There is precedent for this type of approach in design rules and guidance where the nature of the loading is similar to bow slap/slam and green water:

- Rules such as DNV fixed structure and ISO guidelines require both ultimate limit state (10^{-2}) with appropriate safety factors and accidental or progressive collapse limit state (10^{-4}) checks with unit safety factors.
- Code checking for seismic conditions to API rules includes both strength level (elastic design) and ductility level (inelastic response) events.
- Blast wall design is governed by extreme events, since lower level events, even with a safety factor, do not govern.
- It is also noted that the HSE are advocating the use of 10,000 year events for the sizing of air gaps, which is another design condition where step change in loading will occur for very severe events (i.e., rare waves impact the deck, design waves may not).

Not only is it necessary to consider the ALS condition for water impact loads, from observations of damage and test results obtained, it is likely that in many cases this limit state alone will govern the required strength of components. In such cases, it will not be necessary to perform ULS checks. However, ULS design conditions may still govern the design of some components, for example:

- Where other load effects (dead/live load and vessel motion effects) combine with green water loads.

- Where rare extreme wave impact loads are limited by such effects as wave breaking and trapped air, such that extreme loads are not substantially greater than ULS design events.

Results obtained from SAFE-FLOW suggest that ALS criteria will normally govern for green water (and probably water entry) problems due to the very onerous nature of rare extreme events. However, either ULS or ALS conditions can govern for wave impact design, depending principally on the height of the target structure up the bow (higher structures are only reached by more severe events). The suggested methodology is therefore to adopt ALS design criteria throughout, but to check ULS conditions as well where these are likely to govern, based on typical 100-year return conditions and conventional load and material partial safety factors. The ULS check provides a more conventional check that safeguards against such conditions being more significant, but also provides an assurance that the structure will not require excessive maintenance and repair due to repeated moderate water impact events.

3.6 Structural design by detailed analysis

One means by which the designer may demonstrate the integrity of a structure is to assess structural response under factored spatial and temporal variations of pressure acting on the wetted surface of the structure under consideration. Sufficient information is provided in the SAFE-FLOW technical reports to define load time histories that are appropriate to each load type (green water, wave impact, turret impact).

The magnitudes of these load time histories represent single recorded events are do not necessarily reflect the design conditions required for the structure being considered. These design events should therefore be scaled as follows to provide analysis loads for use in finite element analysis (FEA) or other calculations:

- The loads should first be factored to represent the design conditions for the structure being considered, either using pressures on deck target structures from the GreenLab methodology, or using pressures on bow structures from GreenLab.
- The loads should also be factored by a mean load factor as derived from reliability analysis (or taken from SAFE-FLOW, if the structure is sufficiently close to those analysed by the project).
- Where significant for structural dynamics, different immersion velocities should be considered.

Specific rules are presented in appropriate Sections of this report for green water and wave impact.

It is expected that the designer would use this load data to perform transient dynamic analysis whereby the expected spatial and temporal variation of load would be applied to a dynamic simulation of the component. This may be necessary, for example, during the assessment of an existing design which does not satisfy simpler design requirements, or which is otherwise unique.

If design is to ALS criteria, then non-linear, inelastic response is permitted. For such analysis, it is necessary to use a suitable geometric and non-linear analysis program and modelling that simulates the following:

- The as-built geometry of the structure with sufficient mesh density to simulate partial yielding over cross-sections and the buckling behaviour of webs, etc.
- Realistic boundary conditions or modelling of sufficient of the structure such that boundary conditions are not significant.
- The above spatial and temporal variation of load across the structure modelled with sufficiently small time steps to ensure that dynamic effects are correctly simulated.
- Initial imperfections simulated such that the minimum resistance of the structure is obtained.
- Geometric non-linear effects, such as buckling of webs and tripping of flanges.
- Material non-linear effects representing the expected stress-strain characteristics of the material in use.
- Strain rate effects, if used, should be justifiable for the rate of load application considered.

The added mass of water on the wetted face of the structure being considered should be included in this analysis, where this has a detrimental effect on structural natural periods and response. Sufficient text is given in appropriate sections of this report to calculate the effective thickness of water acting with the structure.

It is recommended that strain limits be imposed on structures subject to ALS design events. Such limits should be set to prevent the fracture that can occur due to high strain and possible fabrication imperfections (or fatigue crack growth). In the absence of other information, 5% strain is a suitable limit, consistent with blast wall design. It should be noted that lower values may be appropriate where fabrication quality and fit-up is poor.

3.7 Equivalent design loads

3.7.1 General principle

Design of structures subject to water impact effects requires the assessment of potentially dynamic, non-linear response to spatially and temporally varying loads for one or more limit states. Considering all of the above effects in detailed finite element analysis will result in a complex and potentially lengthy design process.

For practical purposes, therefore, designers require a relatively simple means of determining vessel geometry and member sizes without the need to derive transient loading and perform complex response calculations. This is particularly true of the conceptual and initial design stages, where final geometry is not confirmed and where only preliminary member sizes are required, or where the consequences of selecting different freeboards, for example, needs to be quickly determined.

There are two aspects of design that require simplification if equivalent simplified design loads are to be defined:

- The true spatial and temporal variation of pressure on the face of a structure, including any dynamic response to this load, needs to be represented in a simplified form. This is described using the concept of the Equivalent Design Load in Section 3.7.2.
- If ALS conditions are being considered, then a further simplification of the loading is required to derive an Equivalent Elastic Design Load that, if applied to an elastically designed structure, would produce the same structural design as required by non-linear response to the true loading pattern. This is discussed in Section 3.7.3.

An example of the derivation of equivalent design loads is given in Section 3.7.4

3.7.2 Spatial and temporal variation of load

To simplify structural design, the SAFE-FLOW project includes the generation of Equivalent Design Loads that would produce the same design as the true spatial and temporal variation of load on the structure. Such Equivalent Design Loads are shown diagrammatically by Figure 3-10.

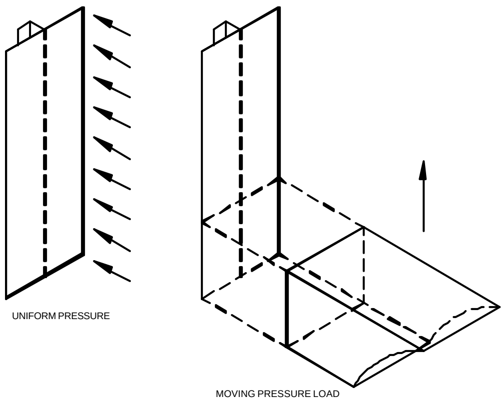


Figure 3-10: Concept of equivalent load design

The magnitude of equivalent design loads for green water and wave impact loading is described in appropriate Chapters of this report. However, the following general comments apply to both loading conditions:

- The equivalent design load will typically vary in magnitude depending on the extent of the structure being designed. The reason for this variation is that local components of the structure could be subjected to localised high pressures, whereas larger components would not be simultaneously subjected to such high sustained loads but would see a lower integrated load. It is therefore important to define the size of the target structure when generating these loads.
- Expected dynamic response of the structure is incorporated into this Equivalent Deign Load so that only simple static analysis of the component with this loading is required.

An example of the derivation of equivalent design loads is given in Section 3.7.4.

3.7.3 Elastic and inelastic design

If design is being undertaken to ALS conditions, then the target structure may be designed inelastically, i.e., the structure may be damaged by the load provided that it remains essentially intact. The potential capacity of the structure beyond first yield is significant, often up to twice the capacity, depending on configuration. This is illustrated for the case of an encastré column in Figure 3-11, below:

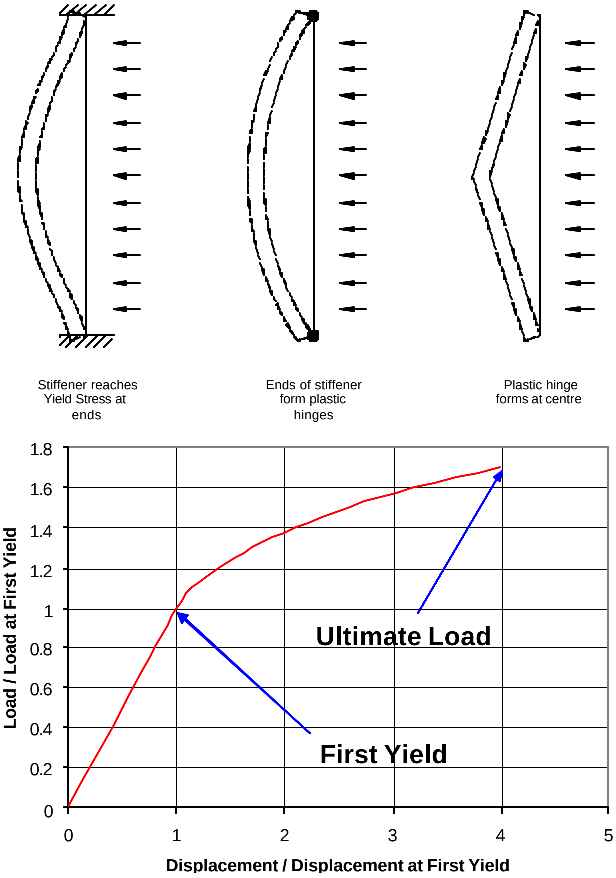


Figure 3-11: Inelastic response of a typical structure

However, inelastic design of structures is significantly more complex than elastic design, requiring lengthier analyses and knowledge of how the material will behave and how the structure will deform (possibly buckle) beyond first yield. Such analysis is normally provided by non-linear analysis programs that can predict post-yield and post-buckling behaviour (see Section 3.6).

As an alternative to this complex approach, SAFE-FLOW allows the use of the Equivalent Elastic Design Load. This is defined as the level of load (typically pressure) such that design of the structure, if performed statically and elastically (so as not to exceed yield), will produce the same structural sizes as would be required to satisfy extreme (ALS) loads, albeit with some damage.

The ratio between the EEDL and the Equivalent Inelastic Design Load (the equivalent design load derived for ALS conditions) is termed the Ultimate Strength Ratio (USR).

It is a measure of the amount of additional load that can be resisted by a structure beyond first yield. This is similar to the Reserve Strength Ratio used for jacket structures, but is defined slightly differently, hence the different terminology. The USR is defined as:

USR = (Maximum load permitted by inelastic design) / (Load at first yield)

so that:

EEDL = EIDL / USR

Typical values of the USR for hull and deck structures are given in appropriate supporting technical documents.

3.7.4 Example of equivalent design loads

As an example of the use of equivalent design loads, consider the case of a deck house column which is fully fixed at its ends and is subject to insignificant axial load. It has been determined that ALS conditions govern over ULS, since there is little or no freeboard exceedance for 10 year conditions. The structure will therefore be designed to ALS criteria, permitting inelastic response. The following calculation procedure is used:

- Determine peak pressure from GreenLab:
  - Design freeboard exceedance for ALS = 8.3 m
  - Height of water at structure = 4.3 m
  - Mean peak pressure on structure = 26.2 kPa
  - Mean load factor from SAFE FLOW = 1.75
  - Design peak pressure =26.2 $\times$ 1.75 = ***45.9 kPa***
- Equivalent Inelastic Design Load
  - Ratio of equivalent load to peak load for stiffener =0.80
  - Equivalent Inelastic Design Load = 45.9 $\times$ 0.80 = ***37.8 kPa***
- Determine Ultimate Strength Ratio for typical stiffener:
  - Pressure at first yield (elastic limit) = 23 kPa
  - Pressure at failure (based on maximum strain) = 53 kPa
  - Ultimate Strength Ratio = 53/23 =2.30
- Equivalent Elastic Design Load
  - Equivalent Elastic Design Pressure = 37.8/2.30 = ***15.8 kPa***

GreenLab is then used to determine the design freeboard exceedance and the mean peak pressure at the target structure location on the deck of the vessel for the worst 10,000 year return sea state. SAFE-FLOW suggests that a mean load factor of 1.75 is required to achieve the required probability of failure for the structure, giving a design peak pressure of 45.9 kPa. However, the Equivalent Inelastic Design Load is only 80% of this value, reflecting the fact that the peak pressure is not felt simultaneously along the stiffener, but allowing for dynamic effects.

The USR is determined for typical encastré beam structures as 2.30, which signifies that the structure can safely carry over twice the load that causes first yield. The structure therefore needs to be designed elastically for as little as 15.8 kPa uniform lateral load over the impact height.

The designer now has the choice of designing the structure anywhere between:

- inelastically (probably by FEA methods) using the full spatial and temporal variation of load and the calculated peak pressure, or;
- elastically, using the Equivalent Elastic Design Load over the loaded height.

Figure 3-12, below, illustrates these extreme analysis methods.

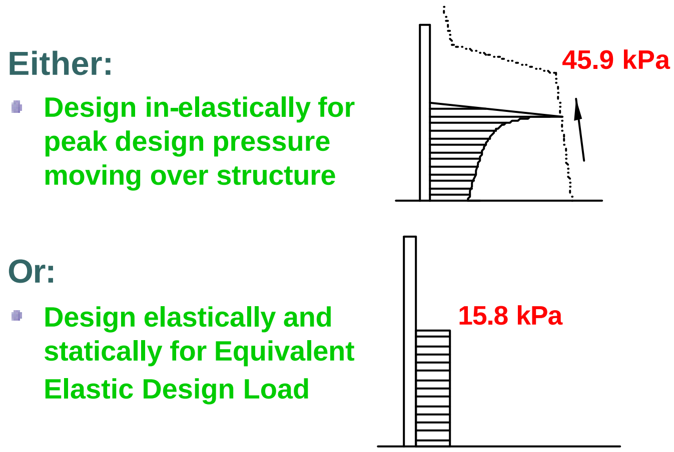


Figure 3-12: Example of equivalent load design

3.8 Other structural design issues

3.8.1 Structural detailing

The full elastic and inelastic capacity of a member can only be relied upon if the structure is 'compact' so that premature failure by web buckling or flange tripping is avoided. There are many requirements presented in structural design rules to define compactness. If these are not met, then additional stability can be provided by the use of web stiffeners or tripping brackets.
If sections are not compact and stiffening is not provided, then detailed analysis is necessary. This detailed analysis should consider different imperfection modes to reduce the capacity of the section, in accordance with Section 3.6, above.

In determining structural capacities, consideration should also be given to other effects that may reduce the capacity of a section, such as quality of fabrication and corrosion.
It is particularly important to consider the above effects if design is being performed inelastically to the Accidental Limit State. Buckling, corrosion and fabrication quality can have a more significant effect at the higher levels of stress associated with these events.

Unless noted otherwise, calculations in the SAFE-FLOW reports assume that the structures are compact, are well fabricated with good quality control and that they are protected from corrosion.

3.8.2 Combinations of load

It may be important to consider water impact loads in combination with other load effects, where this is detrimental to the structure. A particularly important consideration would be acceleration loads (gravity and vessel motion) on deck subject to green water loading. Acceleration load effects on bow structures are unlikely to be significant, but should be considered in special cases where this is not the case.

For ALS conditions, acceleration loads should be assigned a load partial safety factor of 1.0. It will be conservative to assume the maximum vessel accelerations for the green water sea state simultaneously with green water loading. In practice, this conservatism is not expected to be too significant, since the vessel is expected to be bow down and accelerating upwards (adding to gravity loads) at the time of the green water event.

For ULS conditions, consideration should be given to two different combinations:

- Maximum (factored) green water loading added to normal (unfactored) acceleration loads.
- Maximum (factored) gravity loads added to normal (unfactored) green water loads.

The main importance of gravity loading is expected to be its contribution to the buckling of deck house walls, helideck columns and the like. The masses on which such acceleration forces are based should be a reasonable assessment of the likely maximum load in the structure at the time of the impact event, rather than extreme area loads used in design.
Acceleration loads should be omitted where they are beneficial to the structure and where there is some doubt about their magnitude. For example, forward accelerations on forecastle structures that oppose wave impact loads should not be included.

3.8.3 Design for shear

Special consideration should be given to shear loads at the ends of structural stiffeners and girders. The main reasons for this are as follows:

- There is no alternative load path for shear failure. Once this has occurred at the end of a member, failure is likely to follow soon afterwards. There is therefore little redundancy associated with shear failure and the Ultimate Strength Ratio for shear failure will be smaller than for bending.
- Impact loads are potentially more concentrated over one end of member (at the bottom of a deck house stiffener and towards the end of a bow stiffener where wave impact may be concentrated) than over the structure as a whole. This is a function of the spatial and temporal variation of the load.
- Stiffener steel plate structures are typically unbalanced, with a greater concentration of area at the plated side, despite only part of this plate breadth being fully effective. Highest shear stresses for a balanced section are normally at the centre of the section, but for unbalanced sections migrate towards the plate side (see Figure 3-13). High shear in the centre of a section is not normally a concern, as there is little bending at this location, but high shear adjacent to the plated face of a structure combines with high bending and direct pressures due to the impact loading.

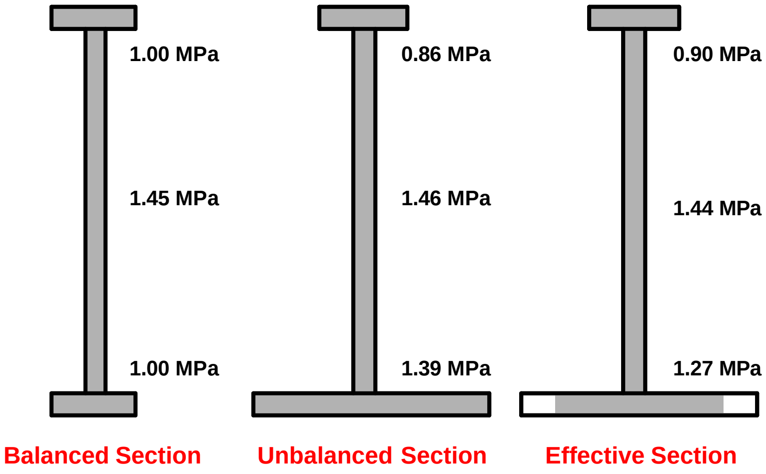


Figure 3-13: Shear stresses in typical stiffened plate sections

The above is entirely consistent with the bow failure on BP Schiehallion, which was initially a shear failure of the web. It was only when support of the web was compromised that tearing of the bow plating occurred. The situation was exaggerated by the relatively poor fit-up of components and the resultant poor weld quality at the end of the member.

It is recommended that shear loading be considered in impact problems as follows:

- Higher Equivalent Design Loads should be calculated over smaller target structures for shear loading than for bending, to allow for the concentration of load at one end of a member.
- Where Equivalent Elastic Design Loads are used, Ultimate Strength Ratios for shear should be applied, which are typically lower than for bending.
- Equivalent (von Mises) stress checks should be performed to evaluate and check combined stress adjacent to the plate.
- Consideration should be given to gussets to reduce shear stresses at such locations.
- The quality of fabrication should be strictly controlled at critical connections likely to be subject to water impact loads.
- Consideration should be given to full penetration as opposed to fillet welds in these areas.

3.9 Fatigue considerations

Fatigue is unlikely to be a design consideration for green water loading, unless this is very frequent. Such cases should be the subject of special study.
Fatigue on hull structures subject to wave impact or water entry are more susceptible to fatigue, particularly where such structures are close to the water surface such that simple immersion pressures add to fatigue damage accumulation. This is discussed in more detail in Section 7.8.

3.10 Weathervaning and heading control

An important issue in assessing susceptibility both to slam loading and green water events is to assess the relative heading of the vessel to the incoming waves. It is consequently important to make a proper assessment of the response of an FPSO's weathervaning characteristics and possible heading control capabilities with respect to the local weather conditions. Several factors contribute to the heading response of an FPSO, specifically, currents, waves (wind driven seas and/or swell), thrusters (if used) and wind. In different circumstances any one or several of these factors can dominate the vessel's heading response.
In lower to modest sea states, a vessel with thrusters may be under the control of the master and the heading can be chosen to suit any operational requirement whether this is to avoid seas from particular directions or to assist with supply or ventilation requirements.

Simultaneous presence of wind-driven seas and swell or current from a different direction may make it difficult to achieve a good compromise for all operational requirements. The consequence of this observation for the present document is that it may be necessary during design to examine all possible combinations of heading relative to wind driven seas and swell for wave heights up to some modest value. It is important to ascertain whether or not any of these combinations can lead to onerous responses and impact load cases.

In the highest sea states the heading of the vessel will largely be under control of weathervaning forces due to the environment. Even if thrusters are fitted, their capacity to resist the highest moments induced by waves will be limited and hence the vessel heading may be expected to be within a small angular range of head seas.

In this case it is important to assess the range of heading angles that the vessel can adopt relative to the incoming waves. This is liable to reveal that the combination of vessel yaw response, wave spreading within the sea state and the other environmental forces (swell, current and wind) will only allow a restricted range of vessel-to-wave heading angles. Typically, for these severe weather conditions the heading-to-waves may be in the range +/- 30 degrees, say, however, it is important to make this assessment for each vessel taking into account the full range of environmental conditions that may occur at the planned deployment site. The majority of the model test conditions investigated within the SAFE-FLOW project were carried out for fixed vessel heading angles so that data could be gathered for a wide range of applications. However, this puts the onus on the designer to make an assessment as described above and select the most appropriate results that are relevant to his vessel.

In moderately severe weather, the picture is more complicated. The yawing moments due to waves may not be so dominant and hence a wide range of vessel headings-to-waves is possible depending on other effects (or the use of thrusters where available). In this case, it would be advisable to investigate a full range of heading angles by model test even though some may be more severe than would normally be considered in design. As found within the Safe-flow project, modest, high-steepness waves can generate very large slamming loads.

If high slam loading occurs at a low level of probability in any sea state, it will be necessary to appreciate the overall duration of these lower sea states throughout the design life of the facility. Normally, moderate sea states will occur more often than those associated with the highest waves. Consequently it will be necessary to consider the model test results carefully to identify whether they contain evidence of high slam loads for waves at the larger heading angles. Such findings may need to be taken into account specifically in the structural design or they may be used to set design requirements for the heading control to constrain vessel response within specified limits.

3.11 Variations in draft and trim

Green water loads and water entry slam loads at any point around the deck are strongly affected by the vessel draft and relative response between the deck edge and the wave surface. To make an assessment of these issues, it is important to appreciate how the vessel will be operated and the portions of time it is liable to spend at different drafts.

Green water events are liable to be most severe when the vessel is at fully loaded draft since this will normally correspond with minimum freeboard and maximum freeboard exceedance. (If the relative motion response peaks at some other draft then it is possible that some intermediate draft may cause the highest freeboard exceedances). In any event it is important to appreciate the probability distribution of draft in making an assessment of green water loads.

Note that for some FPSOs, draft changes substantially depending on level of stored production while for others, draft is held almost constant by compensatory ballast movement. FPSOs may be subject to an operating regime that keeps one tank empty for inspection and maintenance purposes, in which case maximum operating draft will occur less frequently.

In designing for slam due to water entry, it is also important to understand conditions that cause some combination of high frequency of water entry and high water entry velocity of the relevant component (turret or bow flare, say). To assess a maximum design load condition, it will normally be necessary to consider a range of drafts, trims and associated motion responses to derive suitable long term distributions of load and its dependence on water entry occurrence and water entry velocity.

High bow slam loads will be generated by particular configurations of steep wave irrespective of the draft. Vessel draft may affect the effect the elevation on the hull at which maximum loads will be experienced. Measurements from Schiehallion and damage records for other vessels indicate that very high loads can occur at appreciable elevations above mean water level and therefore it is unwise to reduce plate thickness and stiffening arrangements too rapidly with height above the deep draft waterline.

4 GREEN WATER ON FPSO VESSELS

4.1 Introduction

Green water events occur when the relative motion between the water surface and a deck edge or bulwark at a point around the vessel exceeds the freeboard. When freeboard is exceeded by a substantial amount, a rapid flow of water across the deck generally follows and this can cause large impact loads on deck mounted structures, supports and equipment. Consequently it is important to calculate the maximum levels of freeboard exceedance that can occur together with the subsequent water-on-deck flow conditions, to identify whether they can lead to design loads on any such items.

From an operational perspective, it is important to identify the ranges of vessel and sea state parameters (draft, headings, wave heights, wave periods, directions and currents) that give a reasonable probability of moderate free board exceedance so that access to susceptible areas of deck can be restricted when such conditions are forecast.

Bulwarks are often provided toward the bows of a ship to counteract the anticipated, if occasional, large relative motion between the vessel and in-coming waves. Experience with FPSOs suggests that, even when operating with an effective weather-vaning arrangement (that keeps the head into the waves), moderate freeboard exceedances can occur not only at the bow but also rearwards of the mid-ship section as passing waves well-up along the side of the vessel. If the operation of the weather-vaning system is less than perfect - due for example to a strong current, or if there is a swell running transverse to the wind sea then substantial free board exceedances might be expected in other locations.

Hence for an FPSO/FSU operating in any location, it is important to appreciate how local metocean parameters and performance of the vessel heading control may influence the maximum level of freeboard exceedance that might be anticipated around different parts of the ship.

Freeboard exceedance is a highly non linear phenomenon and it has been found necessary in previous analyses of this effect to take into consideration:

- The effect of water on deck.
- The above water shape of the vessel.
- Non linearity's in the waves.

To evaluate forces due to green water, methodologies have been developed in an empirical manner which attempt to address the following phases of the green water event:

- Water heights and velocities on deck.
- Pressure on deck.
- Loading on structures of representative shapes (horizontal and vertical tubulars, support beams and walls of deck houses).

In the context of the calculation of this green water loading, deck equipment is classified as follows:

1. Items of equipment or structures which are large enough to stop or significantly deflect the flow of green water. These structures include deck houses, turrets and large separators.
2. Deck protection structures, such as breakwaters and louvered walls.
3. Items of equipment that are not large enough to significantly modify the flow, including walkways, flow lines, framed support structures.

It is recognised that even small items of equipment can have an effect on water velocities, if these are sufficiently congested to block or funnel flow but, for the sake of limiting the coverage and complexity of the calculations undertaken, effects other than the impact loads of green water on simple structures have not been addressed.
In designing any of these structures, it is important that appropriate structural arrangements be considered and, in the light of the characteristics of the loads that are likely to be experienced, that the models of these structures take into consideration load paths into the main structure of the vessel. As example: the load on a protecting breakwater in front of a turret should be taken by the supporting structure below the deck as well. This can significantly complicate the design of the already complex structure around the turret.
From an overall design perspective, it is important to distinguish items that have a high consequence of failure either with respect to safety, vessel integrity or critical topsides operations. For (heavier items of) deck mounted equipment it is also critical that they cannot be knocked loose and cause subsequent damage to other deck equipment, fixtures or fittings. Where damage may expose a down-flood point (such as the breaking of a venting duct), then it is important to consider the likely influx of water from the initiating and subsequent green water occurrences. Within the context of a safety/risk review, each these issues might be considered for each structure to determine whether they would merit classification as ‘safety’ or ‘business critical components’.

4.2 Nature of green water events

Most of the basic methods used to analyse green water within SAFEFLOW are based on those developed in the previous JIP ‘FPSO Green Water Loading’ and in the PhD thesis of Buchner (2002) to which reference should be made for extensive treatment and discussion. The SAFEFLOW project has extended the model-testing data upon which GreenLab was based and developed methods to provide more detailed consideration of long-term design analysis for green water from the bow, more extensive coverage of green water from the side and development of design methods that incorporate reliability and risk assessment issues.
Observation of green water incidents at model scale indicates that, typically, the following stages are seen (see Figure 4-1):

1. The relative wave motions exceed the freeboard level.
2. The water flows onto the deck.
3. The green water on the deck may form a high velocity water jet.
4. The green water impacts on a structure.

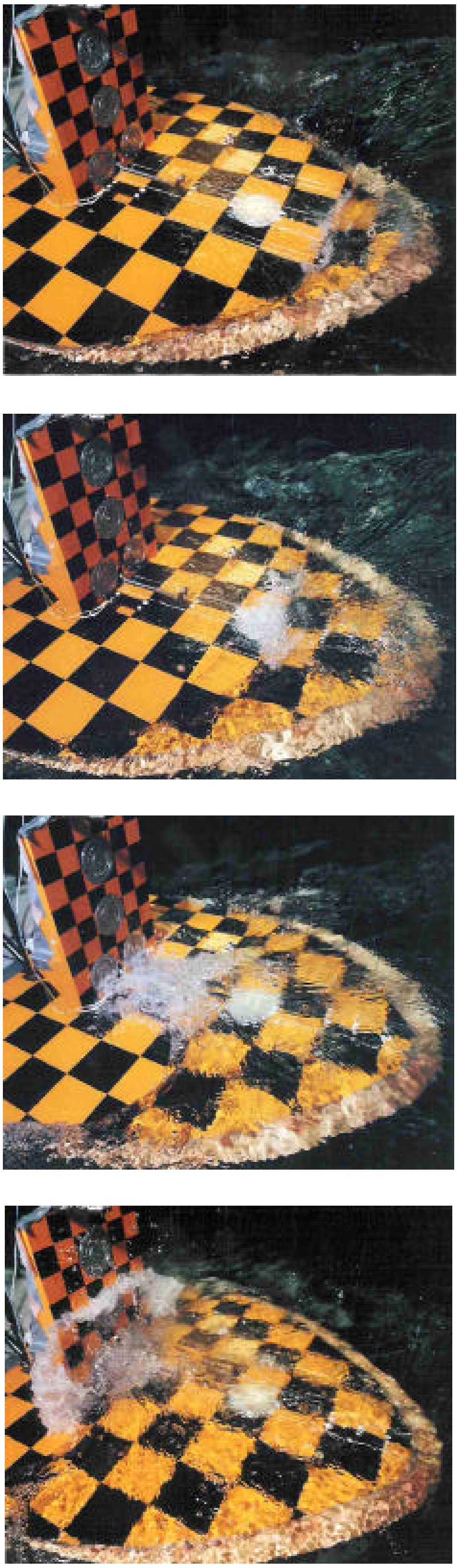


Figure 4-1: Typical sequence during a green water event

Another distinct mechanism for green water loading that was observed during model testing derived from the infrequent occurrence of large steep waves. In some of these cases, a wave crest would break above the level of the bow and send a fast-moving jet of water directly onto the deck or deck-mounted equipment. Both loading mechanisms contribute to the statistical loading distributions used in the methodology that is described in the following sections. In highly unusual environmental conditions it might be necessary to revert to the original data to consider each load mechanism separately and devise an alternative procedure.

4.3 Calculation of relative wave motions

Model test observations suggest that the relative motions around a location of interest on the vessel can be seen as the input for the solution of the green water load problem. The relative motion (r) is defined as the difference between the local vertical vessel motion (z) and the local (disturbed) wave motions (ζ) as follows:

$$r = \zeta - z$$

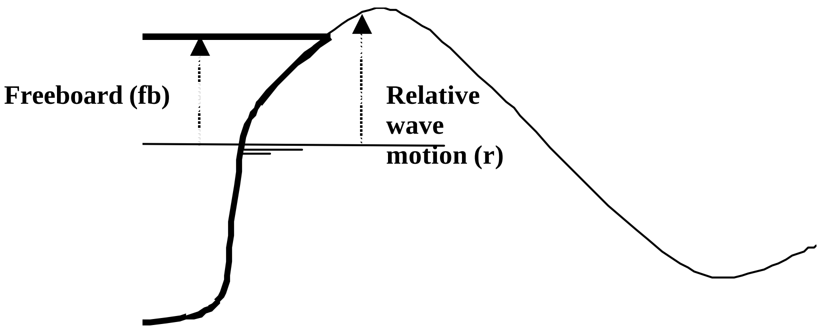


Figure 4-2: Definition of relative wave motion

As soon as the relative wave motions exceed the freeboard level, there is a chance that the green water will flow onto the deck. The freeboard exceedance (h) is defined as:

$$h = r - fb$$

The determination of the relative wave motions requires the calculation of both the ship motions and the (disturbed) wave motions. The calculation of these properties is complicated by the fact that both are subject to substantial non-linear effects.

There are a range of methods that can be used to determine the motions of ships in waves. The simplest and still most widely used is strip theory, a linear 2D method. Strip theory is most accurate for the prediction of the motions of slender vessels but it does not normally account for any disturbance of the water surface caused by the presence of the ship. Given that the calculation of the relative motions requires the calculation of both the vessel motions and disturbed wave surface, and that the vessels of interest are often of considerable beam to length ratio, an analysis based on 3D linear diffraction theory is recommended.

3D Linear diffraction analysis is based on the following assumptions:

- The fluid is assumed to be 'ideal' (viscosity is neglected).
- The waves are considered to be sinusoidal but both incoming and outgoing (diffracted) waves are incorporated.
- The amplitude of the waves and the vessel motions are considered to be small.
- Interaction effects between structure and fluid are only taken into account up to the still waterline.

Based on these assumptions it is possible to linearise the problem. Thus the relationship between the ship or relative motion amplitude and the incoming wave amplitude for any frequency may be expressed as a linear, frequency domain Response Amplitude Operator (RAO).

The assumptions listed above limit analysis to small motions in regular waves, incompatible with the study of FPSO relative motions in conditions in which green water damage is likely. However, it is possible to experimentally determine RAOs, either in regular wave tests as the ratio between the output signal amplitude o_a and the input wave amplitude ζ_a , or from irregular wave tests as the square root of their spectral densities:

$$H(\omega) = \frac{o_a(\omega)}{\zeta_a(\omega)} = \sqrt{\frac{S_o(\omega)}{S_\zeta(\omega)}}$$

Model testing allows responses in realistic waves to be modelled and the non-linearity of the relative motions in increasing wave amplitudes to be estimated. Comparison of the RAOs from regular wave tests carried out by MARIN showed that for identical wave frequencies, the RAOs calculated for different wave amplitude displayed considerable non-linearity. However, analysis of the non-linearity's in the RAOs at particular wave amplitudes is not sufficient to determine the extreme relative wave motions that are required to model the most critical green water events. To incorporate these effects MARIN found it necessary to consider empirical corrections to the short-term statistics of the relative wave motions.

Freeboard exceedance is a highly non-linear parameter and it has been found necessary in previous analyses of this effect to take into consideration:

1. The effect of water on deck.
2. The above water shape of the vessel.
3. Non linearity's in the waves.

4.4 Water flows on deck

4.4.1 Green water over the bow

Green water over the bow results from a combination of the bow moving down and the water surface, modified by diffraction/radiation effects, moving upwards. In addition the wave particle velocities directed towards the bow result in an up-welling in front of the bow. In most of the cases the JIP 'FPSO Green water loading' demonstrated the formation of a wall of water around the bow that upon breaking results in a very rapid flow over the deck. The water particle velocity and the flow depth can also be determined from the JIP results.

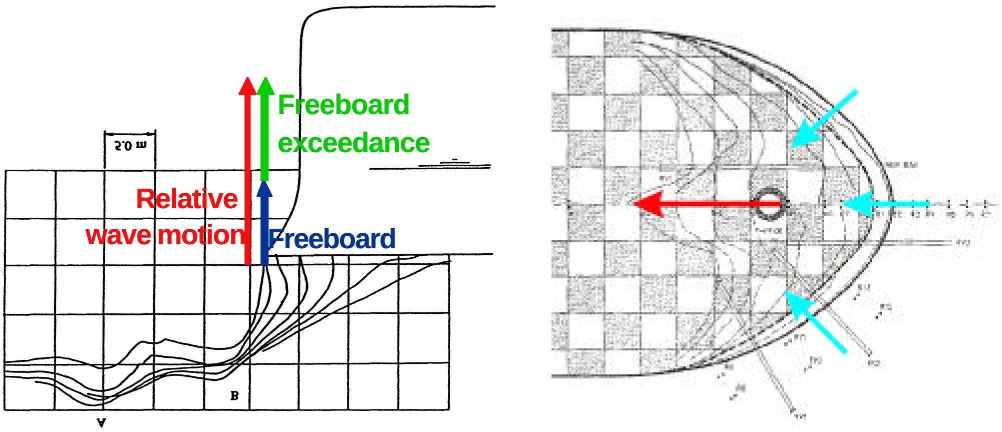


Figure 4-3: Profile view (left) and plan view (right) of green water flow onto deck

Model testing has shown that the flow of water on the deck can be characterised into a number of phases. The following phases can be distinguished in the profile view on the left in Figure 4-3.

Time t	Phase
0.0 s	First an almost vertical wall of water is observed around the bow. The horizontal velocity of this wall is almost zero. The downward pitch angle is at its maximum at this moment.
0-0.5 s	The vertical wall of water translates onto the deck and becomes considerably curved. This gives the impression that it intends to break.
0.5-1.0 s	However, due to the high quasi-static pressure at deck level the water close to the deck starts to accelerate and prevents actual breaking.
1.0-1.5 s	Finally a high velocity jet shoots over the deck. During this process short (non-linear) reflecting and radiating waves slowly propagate away from the bow on top of the incoming waves, the first as a result of the re-entrance of the keel into the water (A) and the second due to the bow flare (B).

Although the flow on deck shown in the plan view on the right in Figure 4-3 depends on the wave period, a typical sequence of phases can be observed:

Time t	Phase
0.0 s	The horizontal velocity of the water front (position of the water wall on the deck) is almost zero.
0-0.75 s	The water front starts to translate onto the deck with a similar velocity from all sides, perpendicular to the local deck contour.
0.75-1.75 s	The water contours from the front and sides meet at the centreline of the ship and result in a water 'jet', which flows with a high velocity aft along the middle of the deck. Typically water front velocities of 15-25 m/s are observed.

The exact flow contours depend on the bow flare angle and the shape of the deck, but the main trend in the observed flow is described above.

4.4.2 Green water from the side

FPSO vessels are often likely to encounter conditions in which there exist non-collinear directions of wind, waves and current. Hence a weather-vaning FPSO will not always encounter head waves. Typically, relative headings between the hull and large incoming waves of 15 to 30 degrees can occur. In such circumstance, green water in the bow area of the FPSO may not be the only problem. The resultant green water from the side of the vessel has caused damage on several FPSOs, typically slightly aft of the midship area.

Although the damage typically occurs to smaller structures (handrails, piping, cable trays, staircases, etc.), it can still cause safety problems for personnel access and damage to fire-water systems. It is also a serious issue for people working on deck, especially since green water from the side can occur unexpectedly in lower sea states.

Video footage taken during model tests of waves along the side of FPSO vessels has shown that the wave crests tend to become more and more peaked as they travel along the side of the vessel. This appears to be a result of the reflection of higher frequency components of the wave along the side of the vessel. In addition, as the wave reaches the midship point, the pitch motions of the vessel may cause the stern to move downwards as well. Thus it was observed that the relative wave motion peaks became so high that they exceeded the freeboard level in the region around midships and slightly further aft. Under these conditions it was noted that the roll motions of the vessel remained very moderate.

When the relative motions exceeded the freeboard, surprisingly fast transverse flows were observed across the deck. The main flow of water on deck was not parallel to the side of the ship, but had a dominant component perpendicular to the length of the ship. The combination of this transverse flow with wave-particle motion is the main loading mechanisms during incidents involving green water from the side. As was noted for green water from the bow, the transverse flow onto the deck bears a similarity to the theoretical dam breaking problem.

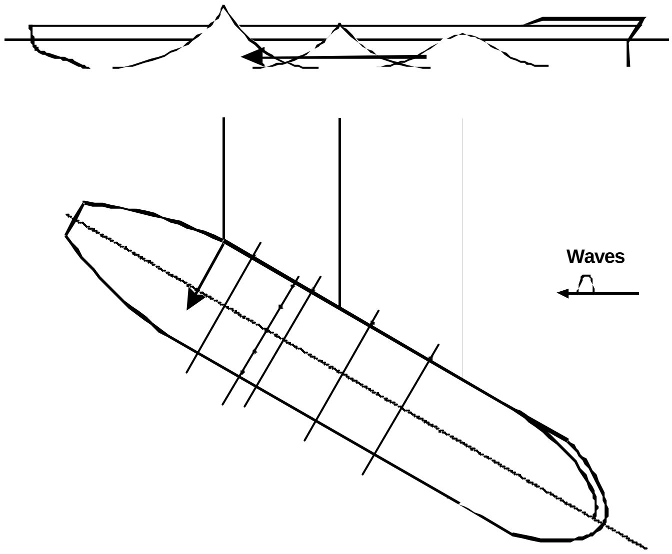


Figure 4-4: Green water along the side of an FPSO

During the model testing carried out in Phase 1 of the SAFEFLOW project, relative motions were determined for four locations along the side of the test vessels, as shown in Figure 4-4. The results of the model tests showed that the relative wave motions along the side of the vessel were significantly non-linear. In addition, the measured number of waves was much larger than would be predicted based on the total test duration and the mean wave period in the wave spectrum. These results tend to confirm the existence of non-linear, higher order effects - such as frequency doubling.

4.4.3 Green water over the stern

Model testing was used to examine the nature of the green water effects at the stern of the FPSO. It was found that these effects were broadly similar though perhaps less dramatic than those observed at the bow of the model vessels. However, since the bulwark at the stern would not normally be as elevated at the bow, substantial green water flows could be generated.

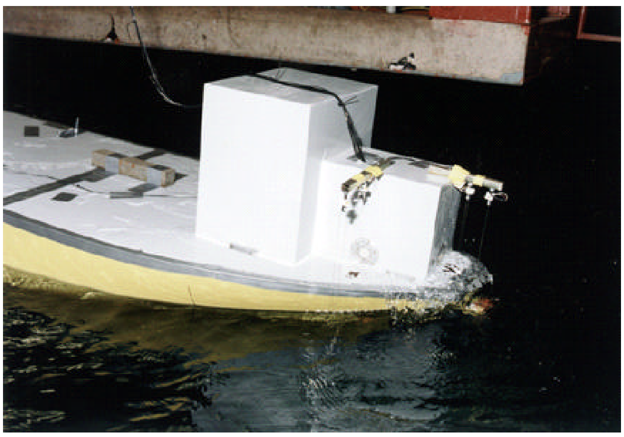


Figure 4-5: A green water event with a traditional tanker stern without poop deck

4.5 Green water impact loading

4.5.1 Introduction

Details of the empirical expressions that have been developed to model distributions of freeboard exceedance are presented in Chapter 5. Having determined methods for the calculation of the freeboard exceedance at locations around the vessel, a means is required to convert these freeboard exceedances into flow conditions on deck and green water impact loads on a range of deck structures. Using experiments as a basis, prediction methods have been devised which take the freeboard exceedance as an input parameter for the calculation of impact loadings.

4.5.2 Green water impact on structures

When a high velocity water front on the deck reaches a structure, a significant impact loading on the structure is often the result. Experiments carried out as part of the SAFEFLOW project have identified the following three stages in the loading of the water on a structure:

- The 'impact stage', resulting in the first and highest peak load (which has a typical rise time to maximum pressure of between 0.10 and 0.35 s). This has the character of an impinging jet or wedge entry.
- The 'quasi-static load stage', which occurs as soon as most of the kinetic energy is out of the fluid and a large amount of water has been built up in front of the structure.
- The 'plunging water stage' when the water built up in front of the structure falls back onto deck. This can result in a secondary pressure maximum.

From observation of the experiment data, it was concluded that the load of the water on deck structures was not due to a solid water-entry impact (as is the case with bottom slamming for example). Rather it was similar to the impact of a jet with an increasing height, as shown schematically in Figure 4-6. This indicates that the time derivatives of the phenomena (dH/dt) will not be infinite, resulting in a wedge angle α significantly smaller than 90 degrees. Therefore, the load may be determined on the basis of a sequence of quasi-stationary loads due to the impinging jet of height H.

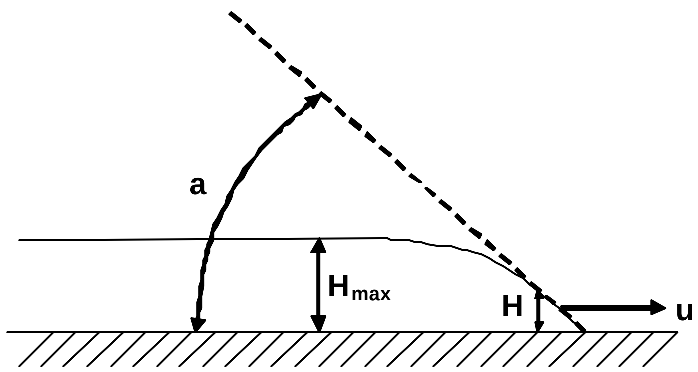


Figure 4-6: Schematic green water flow over the deck

For each time step dt in the initial stage of the impact, the incoming momentum of the water flow is diminished by the impulse of the structure on the fluid according to:

$$F\,dt = m\,dU = m(U-0) = mU$$

Based on the assumption of a constant velocity U of the incoming water flow in the initial stage of the impact and the shallow water assumption of a constant velocity over the full height H, the impulse will be linear with water height. Based on these considerations, it is assumed that the peak force per meter breadth can now be expressed as the rate of change of linear momentum at the moment the maximum water height on deck reaches the structure. This can be written as:

$$F_{Peak} = ?\,H_{Max}\,U^2$$

Experimental measurements have shown that the relation between the water height and the water velocity over the deck is dependent upon the bow shape and the bow flare angle. Hence, a method has been created which defines the relation between the freeboard exceedance and the impact loading on deck structures in terms of the bow shape and flare. The nearly linear relation observed between water height on deck (H) and the freeboard exceedance (h) forms the starting point:

$$H_{Max} = a_H.h$$

The velocity of the water front on deck (U) was found to be proportional to the square root of the freeboard exceedance:

$$U = a.\sqrt{h}$$

As a result the following general expression was arrived upon to describe the pressure acting on a squared structure due to green water:

$$FX = a_F.h^2$$

The coefficients a_p and a_F have been determined on the basis of experimental results and are dependent upon both the bow shape (thin or full) and the bow flare angle (0, 10, 30 and 50 degrees). Tests were carried out by MARIN in both regular and irregular waves. There was found to be a clear relationship between the regular and irregular wave results, the irregular wave results being characterised by greater scatter in the results obtained. To account for the variation in the results obtained in the irregular wave tests, the coefficients a_p and a_F have been determined using two different approaches:

- A value of the coefficients a_p and a_F is determined based on the condition that 95% of the impacts are below the line for p or FX respectively. In this way the model represents a reliability line, which closely models the maximum loads with a high probability and this correlation could be used to obtain a quick estimate of the level of the highest loads that might be expected for a specified set of input parameters.
- An additional value of a_p and a_F is determined based on a least square fit through the measurements in irregular waves. This gives a good indication of the trend in the average load and provides a correlation more suitable for a reliability assessment where several inputs and the load estimation model are associated with significant uncertainties.

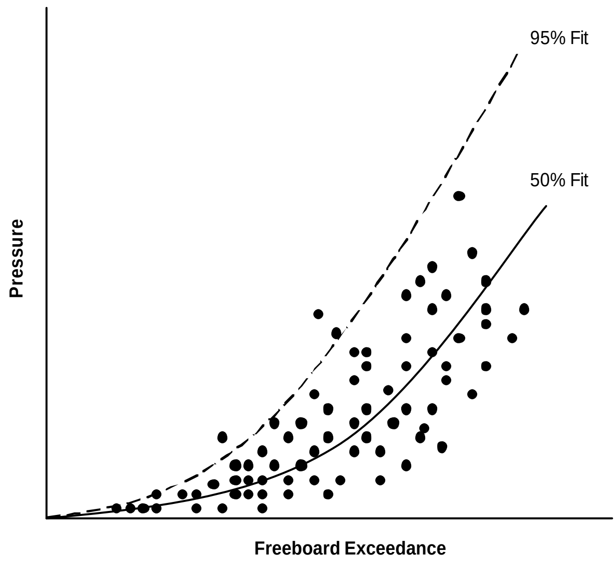


Figure 4-7: Fitting of 95% and least square line to experimental data

The significant scatter in the results reflects sensitivity of the impact loading to small variations in the input. A slightly different water height, water front speed and shape of the water front just before impact can result in large variations in the actual loading.

4.6 Validation with full-scale data

The development of the method for the determination of the long-term distribution of the relative motions provides the potential to identify those sea states that are most likely to cause green water damage. Consider for example the case of green water damage at the bow of an FPSO. Figures 4-8 and 4-9 show comparisons of the recorded conditions in which green water damage incidents have occurred in the North Sea and contour plots of the linear MPM relative motions and percentage contributions to the maximum relative motion respectively from each point in the scatter diagram. The conditions at the time of the recorded incidents, which occurred to a number of vessels, have been normalised on the basis of the ratio of vessel length to wavelength. The return period considered in each contour plot is 1000 years.

It can be seen that the largest relative motions are to be found in the region where wavelength is approximately equal to the vessel length. This region is also that in which the majority of the recorded incidents have occurred. Similarly, Figure 4-9 compares the incident conditions with the contribution each point in the scatter diagram makes to the probability of exceeding the single MPM event in 1000 years. In this case the conditions predicted to contribute most to the probability of the exceeding the MPM relative motion appear to match even better the conditions in which incidents occur, being further inside the outer boundary of the 1000 year scatter diagram. For more details on the methodology used, reference is made to Chapter 5 and the WP4 Technical Reports.

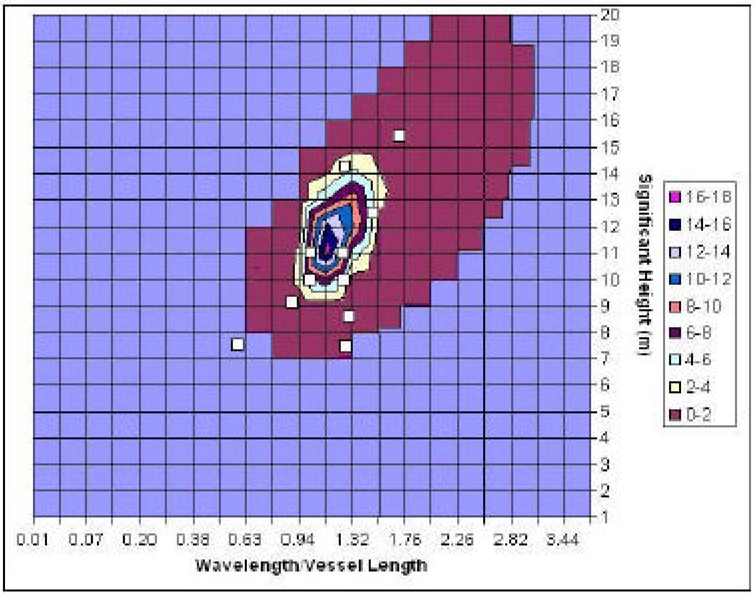


Figure 4-8: Comparison of reported green water incident conditions with predicted MPM freeboard exceedance

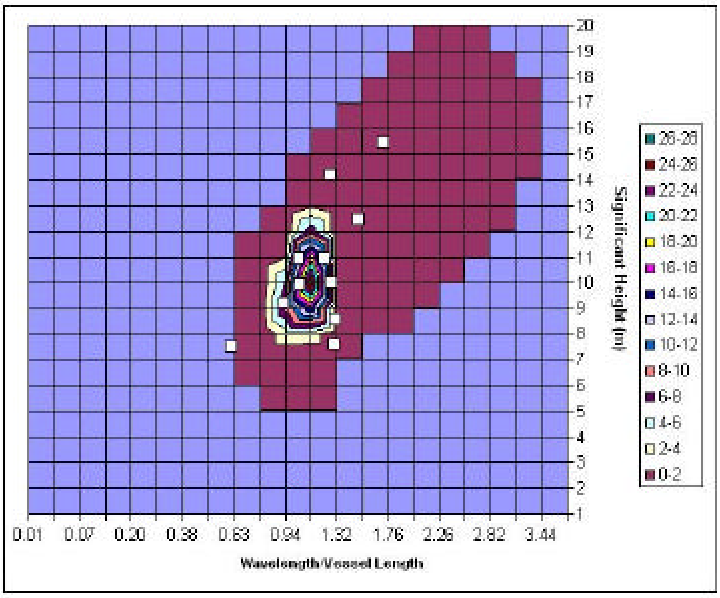


Figure 4-9: Comparison of reported green water incident conditions with contributions to 1000 year MPM freeboard exceedance using all global maxima approach

5 GREEN WATER DESIGN GUIDANCE

5.1 Introduction to methodology

Operational experience has shown that water impact damage to deck equipment on Floating, Production, Storage and Offloading (FPSO) vessels can occur in conditions less severe than would cause an extreme wave or other extreme design loads. For this reason it is important to gain an understanding of underlying contributory mechanisms and appreciate, both in terms of short-term and long-term responses which sea states are liable to cause the observed extreme load cases. Historically, analysis of the long-term distribution of wave height has been used in the offshore industry for the purpose of determining the required deck height for fixed platforms. However, in these cases the extreme wave heights are simply a function of the wave climate. In the analysis of extreme green water load cases, relative motion between the sea surface and deck edge is the crucial parameter and this is affected both by wave condition and the vessel's response to it. Furthermore, the sea state that has the capacity to cause the highest response may occur so rarely that some lower, more-frequently-occurring sea-state carries a higher probability of generating the extreme event in a prescribed design life.

The method of long-term analysis of FPSO response used within SAFEFLOW begins with subdivision of the design period into a number of representative sea states, each of which can be considered as a stationary representation of short-term wave conditions that apply for a short period, typically taken as three hours, although other periods could be adopted if they were thought to give a better representation of a particular wave climate.

As detailed in the following sections of this chapter, predicted responses to short-term wave conditions are subject to empirical correction based on model test results, before they are subtotalled and summed in a particular way to generate long-term distributions. These long-term distributions allow extreme responses to be identified and the distribution of the subtotals according to wave height and period allow the probabilities to be assessed that any extreme value will arise while particular sea state characteristics pertain.

5.2 Adaptation of general methodology to green water

5.2.1 Method 1 - First principles reliability based design

Figure 5-1 illustrates the use of Method 1, First Principles Reliability Design, as applied to green water loading problems. The approach may be used directly to design structure to provide a certain maximum probability of failure, or more commonly to derive mean load factors for subsequent structural design.

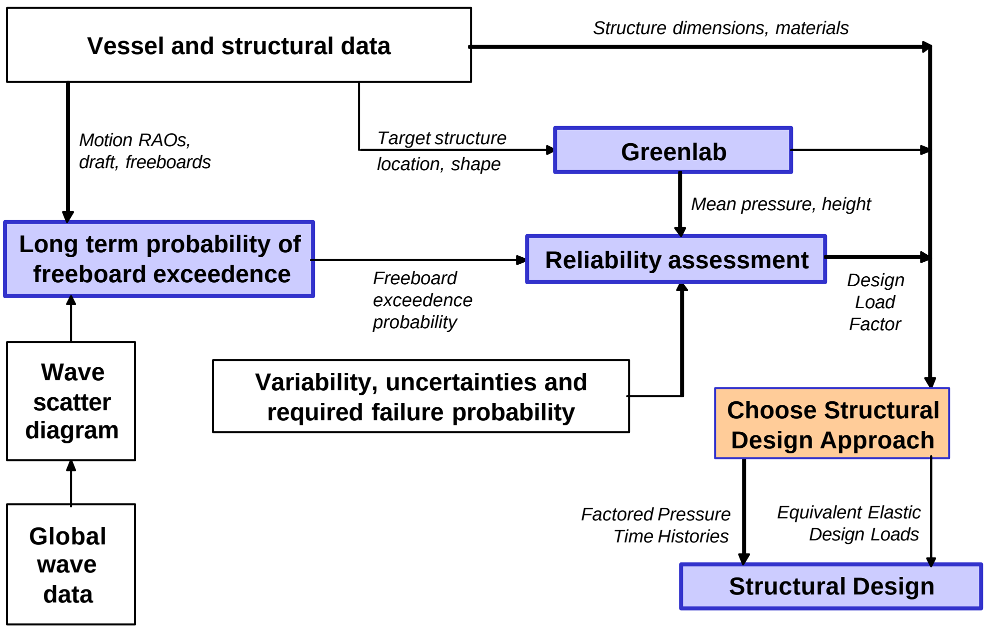


Figure 5-1: First Principles Reliability Design Approach for Green Water

The recommended procedure that should be adopted to design vessel structures subject to green water loading using this approach is as follows:

1. Obtain general vessel data, including operational draft, freeboards at locations of interest (bow, sides, stern) and motion RAOs. Where necessary, multiple drafts can be considered with corresponding freeboard and RAO data. Operating restrictions can be accommodated in the method.
2. Obtain sea state probability data (scatter diagrams) for the likely areas of operation of the vessel, extrapolated, if necessary, to provide suitable resolution in critical seas.
3. Process the vessel and scatter diagram data to produce a long term probability description of freeboard exceedance at each design point.
4. Consider likely target structures in critical regions of the vessel and determine suitable maximum water-on-deck depths and mean ALS pressures or loads from the GreenLab program for the location and shape of the structure. In this step, check that factored ULS conditions do not govern (update the loading if they do).
5. Perform a first level reliability assessment based on freeboard exceedance probability data, target structure pressure data, expected structural and material variability and the required failure probability. Derive from this the expected mean load factor that will give the required failure probability.
6. Apply this mean load factor to the pressure data from GreenLab and perform structural design or strength assessment using one of two methods:
   - Using transient Finite Element Analysis (FEA), or equivalent, determine the response to factored time histories of typical green water loading on the type of structure considered.
   - Determine the Equivalent Elastic Design Load (EEDL) for the structure from the factored GreenLab pressures and the type of structure being considered. Use this EEDL to design the structure elastically.

Note that, for simple structures, it may be possible to use the reliability method directly to design the structure, adjusting the structural size until the required probability of failure is achieved.

5.2.2 Method 2 - Limit state design

Figure 5-2 illustrates the use of Limit State Design as applied to green water loading problems. The approach may be used directly to design structure to provide a certain maximum probability of failure, or more commonly to derive mean load factors for subsequent structural design.

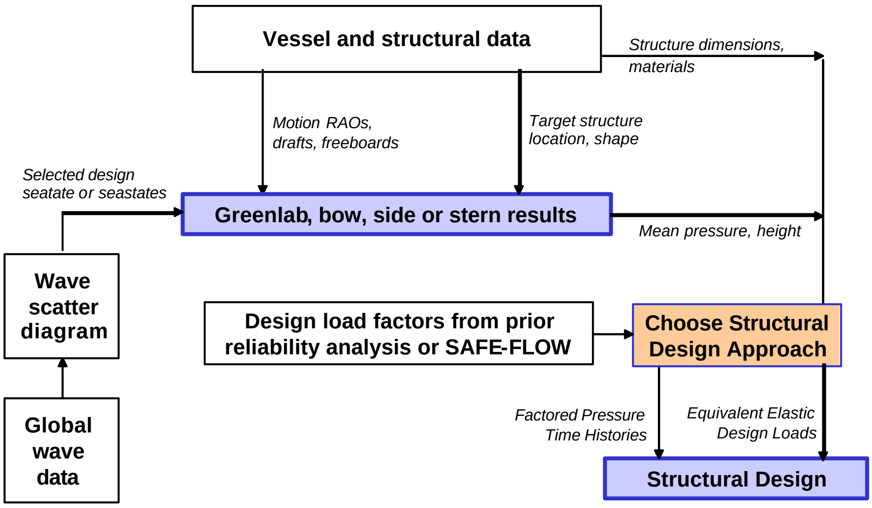


Figure 5-2: Limit State Design Method for Green water

The recommended procedure that should be adopted to design vessel structures subject to green water loading using the Limit State Design approach is as follows:

1. Obtain general vessel data, including operational draft, freeboards at locations of interest (bow, sides, stern) and motion RAOs. Where necessary, multiple drafts can be considered with corresponding freeboard and RAO data.
2. Obtain sea state probability data (scatter diagrams) for the likely areas of operation of the vessel, extrapolated, if necessary, to provide suitable resolution in critical seas.
3. Select appropriate critical sea states and produce most probable maximum freeboard exceedance for ALS design (10^{-4} annual probability of exceedance) using the GreenLab program.
4. Alternatively, process the vessel and scatter diagram data to produce a long term probability description of freeboard exceedance at each design point. Determine from this the design freeboard exceedance with the required annual recurrence interval. This is typically a 10^{-4} event for ALS design.
5. Consider likely target structures in critical regions of the vessel and determine suitable maximum water-on-deck depths and mean pressures or loads from the GreenLab program for the location and shape of the structure. In this step, check that factored ULS conditions do not govern (update the loading if they do).

6. Apply the Mean Load Factor calculated from previous First Principles Reliability Analysis to the pressure data from GreenLab and perform structural design or strength assessment using one of two methods:
   - Using transient Finite Element Analysis (FEA), or equivalent, determine the response to factored time histories of typical green water loading on the type of structure considered.
   - Determine the Equivalent Elastic Design Load (EEDL) for the structure from the factored GreenLab pressures and the type of structure being considered. Use this EEDL to design the structure elastically.

5.3 Calculation of freeboard exceedance

5.3.1 Flow diagram for calculation

The prediction of the long term distribution of green water loads on the deck structures of an FPSO vessel requires data relating to the properties of the vessel, the short-term metocean conditions (sea states, winds, currents, tides) found at the location in which it is to operate and the long-term distribution of metocean conditions that apply to this location. The flow diagram in Figure 5-3 presents the methodology that has been developed for the determination of the long term distribution of freeboard exceedance for an FPSO.

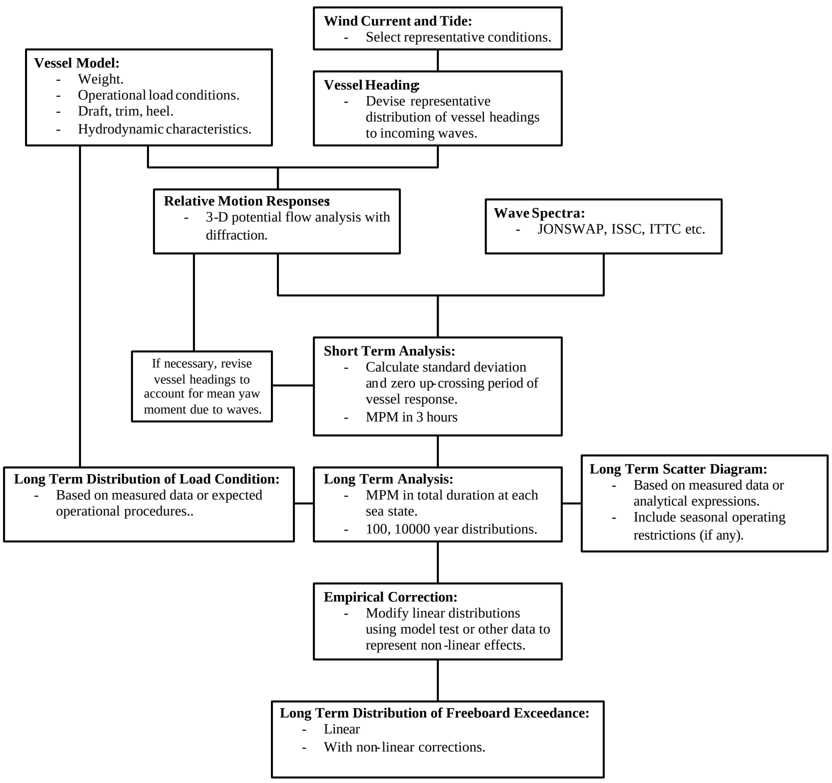


Figure 5-3: Flow diagram of process for determining long term distributions of freeboard exceedance

The flow diagram begins with three separate paths. The left hand side contains the process of defining the vessel model that is to be used for the analysis. This model will contain details of the vessel dimensions, the shape of its hull and the distribution of mass. In addition, the long term distribution of the vessel load condition must be determined. This may be simplified to a finite number of representative load conditions, each with a corresponding probability of occurrence. Secondly, the short term metocean conditions need to be assessed together with details of the mooring arrangement and any operational priorities to establish a correspondingly representative set of vessel mean orientations with respect to approaching waves. In some cases, for example for design against green water over the bow, it may be possible to make a conservative judgement about the response of a turret mooring arrangement and assume that the most onerous wave direction - head seas, say - applies at all times. In others, for example in the analysis of green water from the side, it may be necessary to establish an appropriate probability distribution of wave angles of approach so that several 'stationary' conditions can be taken into consideration in determining the long-term distribution of freeboard exceedance. In the most complex cases, it may be necessary to review the selected vessel headings once the mean yaw moment due to wave/hull/mooring interaction has been established.

SAFE-FLOW has neither researched FPSO load conditions that lead to a distribution of vessel drafts, nor mean responses to currents, winds and waves that might lead to a distribution of vessel mean wave headings, so simplifying assumptions have been made for the purposes of providing illustrative examples. The effects of wave heading have been simplified to the consideration of each of the headings that were investigated in the model test programmes as though each might be representative of a long-term design condition. As indicated previously, applying these issues to any real design may require that more thought is devoted to these issues and the selection of a finite number or distribution of drafts and wave angles on which to base design limit states.

Having specified the characteristics of the vessel, a linear diffraction analysis is performed for the vessel in each of the identified load conditions and headings. This analysis is used to determine the response amplitude operators (RAOs) for the relative motions between the vessel and the water surface at points of interest around the perimeter of the vessel.

The third path relates to the description of the wave spectra that may be used to characterise each of the short-term seas that the vessel is liable to encounter during the design life. A number of formulations are available for the definition of the spectral content of a short-duration, stationary wave system, including the JONSWAP, ISSC and ITTC type spectra. In this study a JONSWAP spectrum has been used. Ochi (1998), describes the selection of wave spectra to represent storm conditions in different areas. Distinctions may be made for benign wave environments, where vessel responses are dominated by other issues, areas in which tropical rotating storms are a prominent feature of the environment, and areas where other storm conditions dominate. Ochi provides appropriate ranges of JONSWAP parameters for the latter two conditions. Alternative methods may be used where measured data are available or more elaborate models are justified.

The standard deviation and zero up-crossing period of the vessel response parameter in particular wave conditions may be determined using the relative motion RAOs and the wave spectra.

The long-term distribution of the vessel response is then determined by accounting for the long-term distributions of the vessel loadings/drafts, heading and wave conditions, the latter in the form of a scatter diagram for the particular location in which the vessel operates. Although, in reality, there may be interdependence among these three factors, for the sample calculations presented in the following, any correlation is accounted in a simplified manner. Empirical corrections are applied to the short-term distributions of the relative motions using model test data. In general, SAFE-FLOW provides empirical corrections based on analysis of three-hour model tests, extrapolated to events that occur more rarely than once in each sea state.
Long-term distributions of relative motion are formed by summing separate distributions representative of the percentage occurrences of short term wave conditions, normally obtained from a scatter diagram. In the examples presented, two sets of long term distributions are created, one based on uncorrected, linear data, the other on corrected data. For convenience of further calculation, two parameter Weibull distributions are fitted to the extreme tails of these distributions although it would also be possible to progress calculations based on the assembled numerical values of the distributions. The Weibull distributions and parameters derived here are used in subsequent reliability analyses.

5.3.2 Calculation of linear relative motion responses

Frequency domain response amplitude operators (RAOs) for the relative motions at various points around the edge of the vessel are calculated using linear three dimensional potential flow methods. These relative motion calculations account for the vessel motions as well as the incoming, radiated and diffracted wave fields. For a particular FPSO, the amplitude and phase of the relative motion RAO at the bow are shown in Figure 5-4. In this case the relative motion is dominated by a peak response around 0.6 rads/s and this is a dominant feature of subsequent analyses. Depending on the precise geometry of FPSO hull this peak may be more or less prominent.

During operation an FPSO would be expected to cycle through a range of different draft and trim conditions. Ideally some attempt should be made to determine the changing nature of these conditions. Such an analysis would need to take into account production rates, ballast handling procedures, offloading cycles, whether some tanks are kept empty for performance of routine maintenance and weather conditions. For the purposes of the analysis presented herein, the long term distribution of the vessel loading condition is considered to be as follows:

Load Condition	Percentage of Lifetime
Fully loaded	10%
All other load conditions	90%

It is assumed that the fully loaded condition represents the most onerous case for freeboard exceedance, in terms of both the lowest freeboard and the largest relative motions. All other loading conditions are considered to contribute negligibly to the largest responses. The draft of the vessel is taken as independent of the wave conditions. Clearly these are idealised assumptions that may need to be considerably revised to represent any particular FPSO. An example can be found in Buchner, Voogt, Duggal and Heyl (2002) for an FPSO in the Gulf of Mexico (GoM).

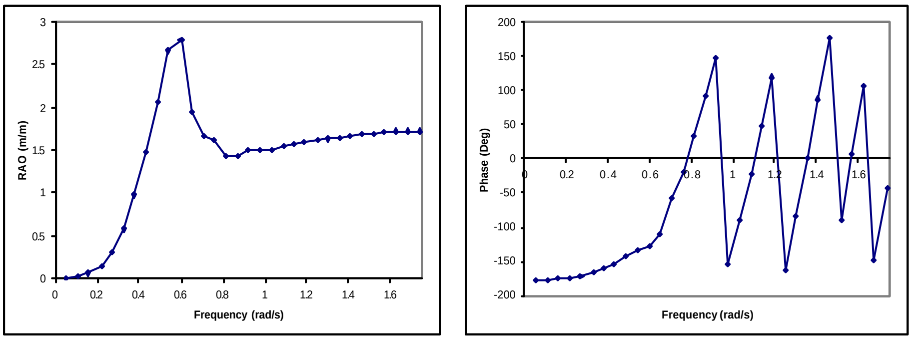


Figure 5-4: Response amplitude operator magnitude (left) and phase (right) for the relative motions at the bow of an FPSO with 30 degree bow flare and an elliptical bow

5.3.3 Long term distribution of wave conditions

In the following example the long term distribution of the short-term wave conditions (sea states) is described using a joint probability density function for the significant wave height (H_s) and peak period (T_p). A scatter diagram has been created using an analytical expression, based on coefficients corresponding to the marginal (all seasons) joint distribution for a typical location in the North Sea.

Hs \ Tp	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	1	1	0	0	0
14	0	0	0	0	0	0	0	0	0	0	0	0	2	6	7	4	1	0	0	0
13	0	0	0	0	0	0	0	0	0	0	0	3	13	24	20	9	2	0	0	0
12	0	0	0	0	0	0	0	0	0	0	3	22	64	79	49	17	3	0	0	0
11	0	0	0	0	0	0	0	0	0	2	28	128	240	212	100	28	5	1	0	0
10	0	0	0	0	0	0	0	0	1	29	207	569	712	464	176	42	7	1	0	0
9	0	0	0	0	0	0	0	1	27	287	1107	1894	1671	855	279	63	10	1	0	0
8	0	0	0	0	0	0	1	28	382	1896	4174	4767	3193	1384	421	96	17	3	0	0
7	0	0	0	0	0	1	32	530	3086	8081	11219	9364	5189	2067	631	156	32	6	1	0
6	0	0	0	0	1	46	807	5057	14549	22889	22396	15046	7501	2951	962	270	68	15	3	1
5	0	0	0	1	76	1364	8579	25313	42232	45503	34945	20621	9908	4055	1464	480	146	42	12	3
4	0	0	1	140	2493	15066	43193	72185	80852	67023	44215	24505	11886	5206	2110	806	295	104	36	12
3	0	2	325	5830	32596	85989	135730	148950	125680	87451	52758	28619	14342	6775	3064	1343	575	243	101	42
2	2	590	11787	61368	145270	208320	213900	174560	121270	75134	42882	23073	11902	5959	2923	1414	679	324	155	74
1	66	3277	22457	55066	74937	71490	54568	36054	21655	12204	6591	3461	1785	911	462	234	119	60	31	16

Figure 5-5: A scatter diagram created using an analytical expression, based on coefficients corresponding to the marginal (all seasons) joint distribution for a typical location in the North Sea

The duration of the analysis has been chosen as 10,000 years. Given that it has been assumed that the vessel condition which contributes exclusively to the largest relative motion events occurs for only 10% of its lifetime, the analysis of the vessel in the fully loaded condition over a 1,000 year duration is considered to be the equivalent of a 10,000 year duration analysis which considers the full rage of vessel conditions. Using the coefficients listed above, the scatter diagram shown in Figure 5-5 may be deduced. The number associated with each H_s, T_p combination represents the total number of three hour events in 1,000 years.

Occasionally, a scatter diagram may be presented to the designer that is based on a relatively short period of observation. Typically, such scatter diagrams will only record occurrences of sea states at probability levels that are greater than 1 in 10,000. If a long-term (1000 year, for example) assessment is required in these cases, it may be necessary to fit the quoted sea-state occurrences to appropriate distributions in order to enable the scatter diagram to be extrapolated and take into account the extreme but rare sea states that occur only a few times in the entire return period.

5.3.4 Short term analysis of relative motion responses

The standard deviation and zero up crossing frequency of the relative motion responses may be determined using the following relationships:

$$m_0 = \int_0^\infty S(w)\, \left| A(w)^2 + B(w)^2 \right| dw$$

$$m_2 = \int_0^\infty w^2\, S(w) \left| A(w)^2 + B(w)^2 \right| dw$$

where A(?) and B(?) are the real and imaginary parts of the relative motion RAO respectively and S(?) is the wave spectrum. The standard deviation (s) and zero up crossing frequency (?$_0$) of the vessel response are thus determined:

$$\sigma = \sqrt{m_0}$$

$$\omega_0 = \sqrt{m_2 / m_0}$$

By assuming that the distribution of the relative motion responses takes the form of a Rayleigh distribution, the most probable maximum (MPM) in a standard three hour time period can be calculated using the following equation:

$$MPM = \sigma \sqrt{2 \ln \left(\frac{3*3600}{T_0} \right)}$$

where T_0 is the zero up crossing period. Figure 5-6 shows the MPM freeboard exceedance (relative motion amplitude minus freeboard). Superimposed upon this figure is a line corresponding to the conditions for which a single three hour event would be expected in the given return period. It can be seen that the largest freeboard exceedances are outside of the single event line. The largest freeboard exceedance predicted to fall within the single event line is just over 12 metres.

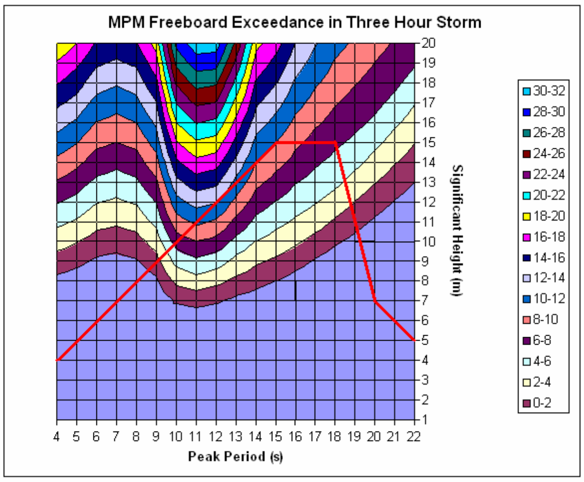


Figure 5-6: MPM freeboard exceedance in three hours at each point in the scatter diagram

However, given that information is available from the scatter diagram about the proportion of the chosen return period that is spent at each particular wave condition, it is more accurate to determine the MPM freeboard exceedance using this data.

The total number of hours at a particular condition in the chosen return period is given by n_{hours}. For example, at H_s = 4 m and T_p = 8 s, the total duration during the period considered is 43193*3 hours, and now:

$$MPM = \sigma \sqrt{2\ln\left(\frac{3600 * n_{hours}}{T_0}\right)}$$

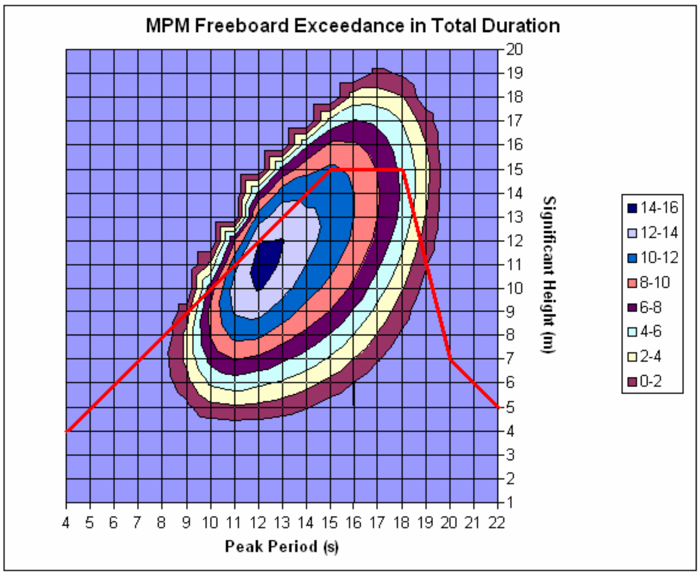


Figure 5-7: Most probable maximum relative motion response for entire period at each point in the scatter diagram

Figure 5-7 shows the predicted MPM relative motion responses at each point in the scatter diagram for the entire duration of those conditions in the chosen return period. The largest value is now 14.7 metres freeboard exceedance.

5.3.5 All global maxima approach to determine long term distribution

As noted earlier, for each H_s and T_p combination in the scatter diagram is it possible to determine the MPM relative motion for the entire duration at that sea state. However, this method does not lead to the overall maximum associated with the entire scatter diagram because it fails to determine the higher values that arise when the total numbers of waves are taken into account for the entire scatter diagram. The all global maxima approach attempts to take this into consideration. This technique involves the determination of a long term distribution of the relative motions which accounts for the contributions due to all sea states. The following steps describe the implementation of the all global maxima approach:

Step 1: For each point in the scatter diagram calculate the total number of relative motion cycles that occur in the total time for those conditions:

$$n_{H_s,T_p} = \frac{a_{H_s,T_p} \times 60 \times 60}{T_{0_{H_s T_p}}}$$

Sum the number of cycles for all points in the scatter diagram to obtain the total number of response cycles in the defined return period:

$$n_{Total} = \sum_{H_s} \sum_{T_p} n_{H_s,T_p}$$

Step 2: For each sea state, calculate the value of the cumulative probability distribution of the relative motion response for a range of response amplitudes, *x*:

$$F(x) = 1 - \exp\left(\frac{-x^2}{2\sigma_{H_s,T_p}{}^2} \right)$$

Step 3: Weight the cumulative distribution for each sea state based on the proportion of the total number of wave cycles in the entire return period that occur in that sea state:

$$F_{H_s T_p}(x) = F(x) \times \frac{n_{H_s T_p}}{n_{Total}}$$

Step 4: Sum the weighted contributions of all points in the scatter diagram to give an overall cumulative distribution function representing the long term distribution of relative motion response for the entire return period:

$$F_{Total}(x) = \sum_{H_s} \sum_{T_p} F_{H_s T_p}(x)$$

Using the cumulative distribution determined using the described steps, the magnitude of the largest single event can be determined based on the probability of exceedance being:

$$1 - F_{Total}(max) = \frac{1}{n_{total}}$$

Having determined the expected largest single event in the defined return period, the contribution of each of the sea states to the probability of exceeding this value may also be calculated. The contribution of each sea state is calculated using the proportion of the probability of exceeding the largest single event due to the weighted individual contribution:

$$c_{H_s,T_p} = \frac{F_{H_s T_p}(max)}{F_{Total}(max)} \times 100\%$$

In addition to being able to be used to determine the largest response maxima in a particular time period, the long term distribution $F_{Total}(x)$ can form the basis of a reliability analyses. In the later case an analytical expression for the distribution is used to represent the data series. This is obtained by fitting a Weibull distribution to $F_{Total}(x)$.

5.3.6 Incorporation of model test results

Linear diffraction analysis commonly overestimates the magnitude of vessel motions in large waves. This is in part due to the fact that they only consider the portion of the hull up to the calm waterline. Vessel features over the still waterline such as overhangs or flare will have the effect of reducing the vessel motions, as well as changing the nature of the radiated and diffracted wave fields. To correctly determine what effects these vessel features have on the relative motions requires either a non-linear analysis or model testing.

MARIN has carried out a large program of tank testing on a range of FPSO models in random seas in the JIP 'FPSO Green water loading' and the SAFE-FLOW JIP. From this data they have calculated a series of empirical correction factors, which, when applied to the Rayleigh distributions of the response determined using diffraction analysis, produce results which closely match experimental results. These experiments have been carried out for:

- Wave spectra with peak periods of 10, 12, 14 and 16 seconds.
- Three bow types (triangular, elliptical and cylindrical) and 2 stern types (traditional ship type and barge type).
- Bow flare angles of 0, 10, 30 and 50 degrees for most of the bow.

MARIN has developed a method by which modified Rayleigh distributions may be used to account not only for the non-linearity's observed in the short-term distribution of the responses, but also for the discontinuity that occurs as the freeboard is exceeded. The short-term distribution of the linear relative motion responses is used as the basis for this modified distribution.
The modified short term distributions of relative motions that do not exceed the freeboard is determined using:

$$F(x > R) = \exp\left[\left(-\frac{fb^2}{2s^2}\right).\left(a + b.fb + c.fb^2\right)\right]$$

whilst for relative motion responses that exceed the freeboard height, the following equation is used:

$$F(x > R) = \exp\left[\left(-\frac{fb^2}{2\sigma^2}\right).\left(a + b.fb + c.fb^2\right) + \tau\left[(R\text{-}fb).d + (R\text{-}fb)^2.e + (R\text{-}fb)^3.f\right]\right]$$

The parameters a, b, c, d, e and f are determined using model test data. The parameter t is determined using the freeboard of the vessel (f_b) and the response standard deviation as follows:

$$\tau = 1.0 + 0.83.\left(\frac{fb}{\sigma}\text{-}1.3\right)$$

Examples of the measurements and modified Rayleigh distributions are given in Figure 5-8.

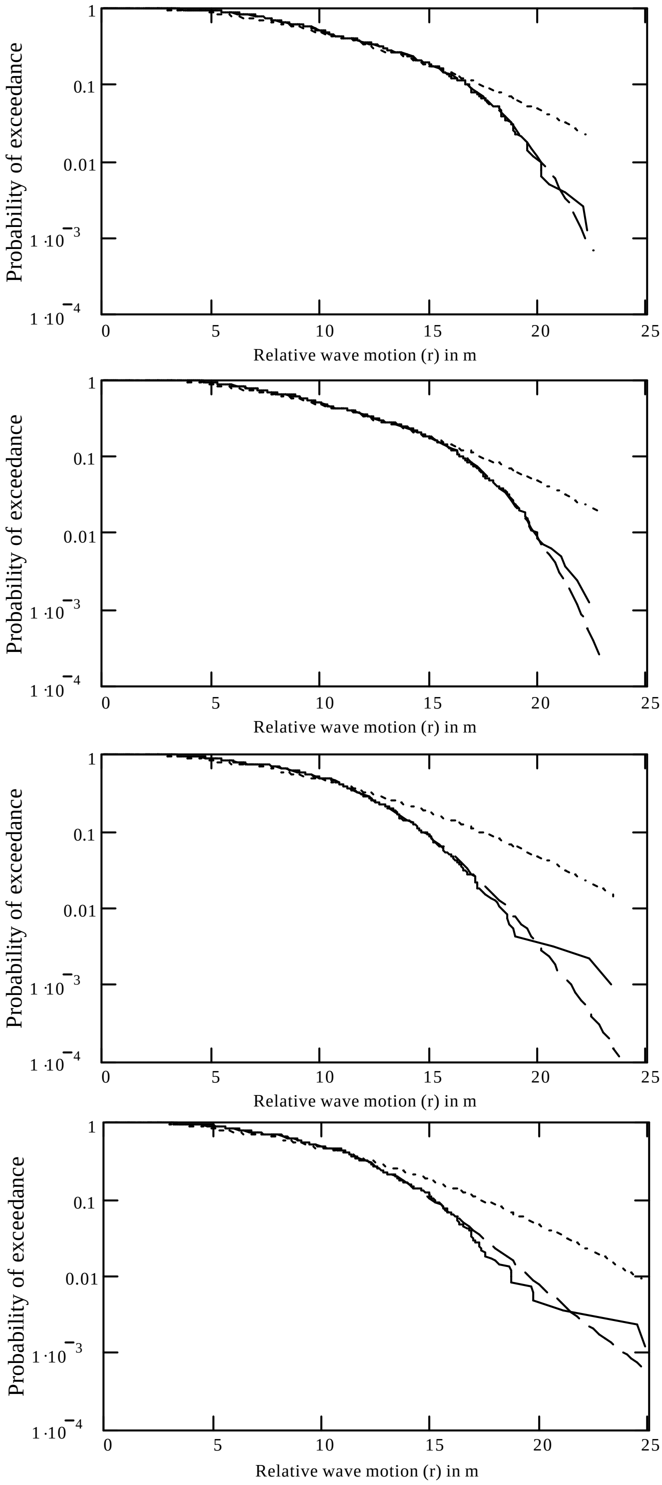


Figure 5-8: Measured probability of exceedance curves with the fitted modified Rayleigh distributions for the full elliptical bow with traditional stern, spectral peak period of 14 s and bow flare angles of 0, 10, 30 and 50 degrees (from top to bottom). The figure shows: the measurement (solid line), the Rayleigh distribution (dotted line) and the modified Rayleigh distribution (dashed line)

Tabulations of all values for each model are provided in the WP2 Technical Reports for new data obtained within SAFEFLOW and in Buchner (2002) for earlier tests. Further discussion of the range of applicability is given in Buchner (2002) and in the WP4 Technical Report.

The resulting modified Rayleigh distributions can be used to in the All Global Maxima approach to determine the MPM non-linear relative motion in a given period. This calculation is complicated, slightly, by the fact that the non-linear corrections available from MARIN were determined with a H_s of 13.5 m and specific values of spectral peak period, namely 10, 12, 14 and 16 seconds so some assumptions must be made regarding how these might best be applied to other sea states.

To ensure accuracy for sea states with high values of significant wave height, the same set of non-linear corrections is applied for all wave heights. Sea states with spectral peak periods less than or equal to 8 seconds do not occur for very high significant wave heights so, non-linear corrections have not been applied to these predictions. For sea states with peak periods greater than 16 seconds, the non-linear corrections applicable to the model tests in the 16 second sea have been applied. For sea states with period between 8 and 16 seconds, an appropriate set of corrections is applied based on the measured set of values that is judged most closely to represent the case being considered.

Some care is required in applying the above method to sea states with extremely large probabilities of occurrence since they may be applied well beyond the range of parameters determined in model tests. Low probabilities of freeboard exceedance, less than 1 occurrence in 10,000 waves, for example, may be highly significant to sea states that occur frequently. Some of the cumulative distributions associated with the above expressions do not converge to unity at low probabilities as would be required for a valid probability distribution. To overcome this problem, the extension of the above expressions to low probability levels, was achieved by requiring that they take the form of a Weibull distribution. The parameters of these Weibull distributions were chosen for each short-term distribution to ensure continuity of values and slopes at a probability of 1 in 1000, which is approximately the lowest probability level that might be expected in a three hour sea state. Where only 3 hour model test results are available, such an extrapolation is fully consistent with all available data and ensures consistency with logical requirements at all probability levels although it must be admitted that the lower probability levels are pure conjecture at the lowest levels of extrapolation. Where longer tests have been performed, checks for consistency/convergence of the above procedure have been applied and, shown to be satisfactory (see the WP4 Technical Report).

When all contributions to the freeboard distribution have been assembled for all the short-term sea states in the scatter diagram, these are combined to form a long-term distribution. Assuming that N-million waves occur in the lifetime represented by the scatter diagram, the extreme value that is expected to occur once in these N-million occurrences can be read directly from the from the assembled distribution. For convenience in later calculations, a two parameter Weibull distribution is fitted to the tail of the combined distribution for response amplitudes in the region of the 1000 year all global maxima MPM. The 1000 year MPM (calculated using the all global maxima approach) and Weibull parameters for the complete scatter diagram and single column of data are as follows.

5.4 Reliability based design calculations

The long term distributions of the relative motions at locations around a vessel form the basis of the reliability analysis of deck structures and equipment subject to green water loading. The use of a first principles reliability based design method allows the strength of the deck components to be set so that they reach a given level of reliability i.e. the annual probability of failure is below a specified value. As noted in Chapter 3, reliability based design is useful in cases where there is no prior history of load factors or material factors.

At its most basic level the analysis of the reliability of structures subject to green water loading can be seen as the determination of the degree by which the likely strength of the structure exceeds the likely loads acting upon it. This balance between the loads and the resistance is expressed in terms of a Limit State Equation as follows:

$$Z = R - L$$

Where R is the capacity of the structure to resist the load acting on it (the resistance) and L is the load acting on the structure. Three possible conditions exist, the first being the load exceeding the capacity ($Z \leq 0$), the load equalling the capacity ($Z = 0$) and the load being less than the capacity ($Z \geq 0$). The objective of a first principles based reliability method is to ensure that the final condition is met, with the degree by which the load capacity exceeds the load defining the reliability of the system.

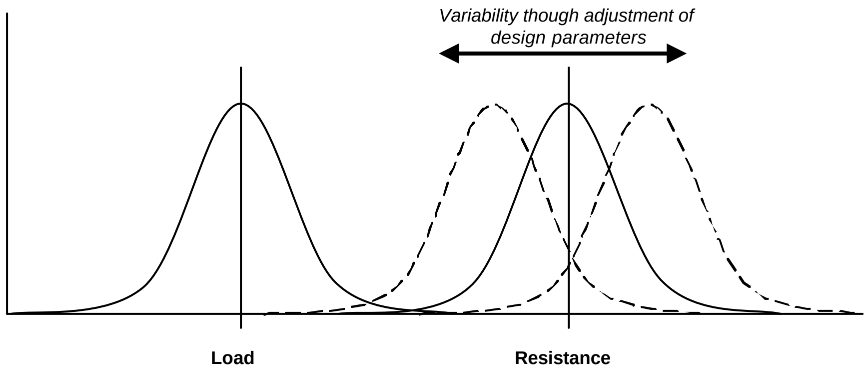


Figure 5-9: Distributions of the load and resistance that make up the limit state equation

The development of statistical descriptions of the load and the strength properties of the structure provides the opportunity to target a particular level of reliability through the design of the structure in question. As shown in Figure 5-9, the manipulation of the structural properties has the effect of moving the distribution resistance either closer or further from the load distribution, decreasing or increasing the reliability respectively.

5.4.1 Determination of green water load distribution

For the purposes of implementing the reliability analysis, the distribution of pressure due to inflowing green water of a particular is modelled as a log normal distribution. As noted in the previous chapter, modelling of the loads on deck structures due to green water is based on simple equations which use experimentally derived factors to calculate the loads from the level of freeboard exceedance.
For each type of structure, two parameters are used, a least square (50%) fit and a 95% reliability line. On the basis of the ratio of these two values it is possible to determine a lognormal distribution which reasonably accurately describes the distribution of load due to a freeboard exceedance of a particular value.

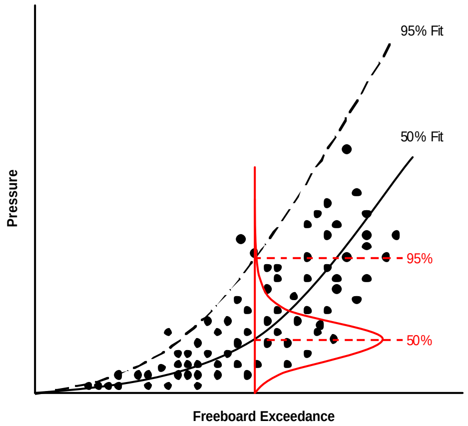


Figure 5-10: Fitting of log-normal distribution to experimental data relating freeboard exceedance to loads on structures

A single distribution for the maximum pressure is able to be determined on the basis of the joint probability distribution of the level of freeboard exceedance and the pressure that results from a particular level of exceedance. The details of this calculation are detailed in the worked examples of this calculation in the accompanying Technical Report.

Having determined a statistical representation of the likely pressures on the structure of interest, a modelling uncertainty factor is included, which accounts for uncertainties related to both the load and resistance components of the limit state equation. These uncertainties include those that arise from model test results and a range of other considerations, including the use of scatter diagram data to represent the wave conditions and uncertainties related to the strength of the structural components.

5.4.2 Determination of structural strength distribution

The variation of the strength of deck structures subject to green water loading should follow a fairly conventional reliability analysis approach, considering and combining uncertainty in various design parameters to produce the overall probability that a given structure can resist a particular applied load or pressure.

Design quantities that may be considered variable for deck structures subject to green water loading include the following:

- The material yield strength (for which a bias and a standard deviation are normally defined).
- Member dimensions and sizes, particularly thicknesses of plate.
- Imperfections and tolerances.
- The buckling strength of an axially loaded member or component.
- Effective breadths of plate and effective widths of flanges.
- Any other loads on the structure, such as axial load due to accelerations.
- The dynamic response of the structure.
- Errors in calculation, end fixity, etc.
- For ALS design, the redundancy and Ultimate Strength Ratio for the component.

In reality, many of the above uncertainties may be grouped into a single variable parameter, typically modelling uncertainty. Further details of typical values used are presented in the accompanying Technical Report on Green Water Design Guidance.

5.5 Derivation of mean load factors using Method 1

Method 1 of the Green Water Design Guidance (a first principles reliability approach) can be used to derive mean load factors for subsequent strength checking. The evaluation of load factors was implemented using the mathematical spreadsheet software Mathcad.

Spreadsheets have been developed for a range of deck structures, including windows on the front of accommodation blocks and breakwaters. Details of these spreadsheets are given in the Green Water Design Guidance Technical Report. In accordance with the approach described earlier in this Chapter, each spreadsheet performs a first principles reliability analysis on the structure in question, then determines an appropriate mean load factor that can be used in a limit state design method.

This mean load factor is defined as the factor applied to a simple design load check that would give the same design that is needed to achieve the target reliability (probability of failure).

5.5.1 Results of reliability analysis for a simple structure

The following results are obtained from reliability analysis of a structure on the front face of an accommodation block. Details of this method are given in full in the accompanying Design Guidance Technical Report. The precise nature of the calculation will vary between structures depending on the nature of the green water loads acting on then and the relationship between the load and the freeboard exceedance. However, results presented here are typical.

For design purposes, it is necessary to know not only the load factors themselves, but also the return period of loading they should be applied to and what level of design they should be used for. In this instance:

- The mean load factors are derived on the basis of a 1,000-year ‘all global maxima’ design freeboard exceedance, and should only be applied to identical events.
- The mean load factors are calculated on the basis of elastic design principles for this design event and factored loads should therefore be used for elastic design.
- Strictly, the factors apply to a window bolt component with a reserve strength ratio of about 1.2 and a material yield bias of 1.10, but sensitivity analysis reported below considers different values.

The 1,000-year design basis used for these calculations has been chosen to ensure a reasonable level of design load from which a relatively stable load factor can be produced. As discussed in Chapter 3, too low a return period event may only marginally exceed the freeboard, resulting in artificially high mean load factors. A 10,000-year design event may be preferred, since this maximises the area of structure affected by green water without increasing the design loading (since the mean load factor would reduce to give the same probability of failure for the more extreme design loading).

The ‘all global maxima’ has been selected as the design basis. In accordance with Section 5.3.5, this is the freeboard exceedance based on risk of exceedance over the entire scatter diagram. It is realised that this may be difficult to obtain in the early stages of design. An alternative approach may be to determine the design basis from selected critical sea states, perhaps following the contour of sea states with a required recurrence interval (say once in 100 years). If this were done, different mean load factors would be required to calibrate the method for this design basis.

To establish the characteristics of the reliability model, sensitivity calculations were performed for some of the key input parameters and the results of this study are summarised in the Table below. The sensitivity analysis was performed by varying each parameter in turn away from one standard input set. The values used as the standard input and the mean load factor obtained with this set are denoted in bold.

Parameter varied	Range of variation	Range of elastic mean load factor required to satisfy design requirement
Ship design life (years)	**20** - 30	**1.399** - 1.385
Material yield strength (MPa)	400 - **550**	1.41 - **1.399**
Standard deviation of Yield strength (kPa)	**32** - 40	**1.399** - 1.406
Coefficient of variation of modelling uncertainty	0 - **0.8** - 0.16	1.381 - **1.399** - 1.452
Required annual probability of failure	10^{-2} - $\mathbf{10^{-3}}$ - 10^{-4}	0.794 - **1.399** - 1.904
Ultimate strength ratio	1.0 - **1.2** -.1.4	1.68 - **1.399** - 1.20

The model is seen to be robust to variations in design life, yield strength, variation in modelling uncertainty and annual return period. For example, there is only a 3.8% change in required safety factor with a doubling of modelling uncertainty.

On the basis of these sensitivity calculations, it would appear that adopting a safety factor of approximately 1.40 would cover most eventualities and therefore might be adopted as representative of this type of approach to design for green water.

The method is seen to be very susceptible to the required annual probability of failure. In accordance with Chapter 3, a 10^{-3} annual probability would be acceptable for most structures when there is forewarning of load (sea states are high), providing that failure is not of high consequence. If this structure located on the front face of a living quarters, then the 10^{-4} failure probability may be more appropriate and the mean load factor would rise from 1.40 to 1.90.

Figure 5-11 shows this variation in the safety factor for a range of target annual failure probabilities. Also shown on this plot is the load factor applied to the 95% fit to the scatter of experimental pressure data. Load factors applied to these higher predicted pressures are lower, to give the same probability of failure. It is recommended, however, that Mean Load Factors (MLFs) applied to the mean of the experimental data be used in calculations, to avoid confusion.

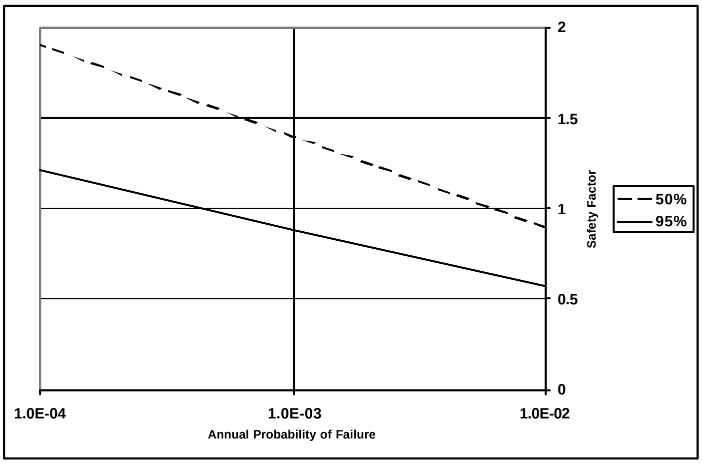


Figure 5-11 Comparison of required safety factor with annual probability of failure

In summary, similar structures on similar vessels in similar environmental conditions should be designed elastically (up to yield) under pressure loading derived from all global maxima 1,000 year return conditions with a load factor of 1.40 if the consequence of failure is not severe, and 1.90 otherwise.

The lower value is not dissimilar to common ULS design criteria, which would suggest a combined safety factor (load x material) of 1.3 x 1.15 = 1.50 for 100-year design events. The factor is a little higher, but the design event lower, so the difference is not expected to be great.

However, the above ULS criteria would normally be applied to manned structures where the consequence of failure is high, but warning of failure may still be expected. Consequently these ULS criteria would be expected to be more consistent with the higher 1.90 factor for green water loading. This is clearly somewhat more conservative than common ULS design would dictate.

The probable reason for this increase in load factor for green water loading is the uncertainty in design pressures reflected by the scatter of pressure data from the Green water JIP. This scatter in experimental data requires an increase in the safety factor to ensure the same probability that design resistance is not exceeded by the more extreme loads.

The same model can be applied to other types of structure that are subject to similar green water loads (i.e., that are dependent on the square of the freeboard exceedance) by first establishing the appropriate ultimate strength ratio between acceptable plastic failure and a failure corresponding to first yield. Given a higher value of ultimate strength ratio, one would expect that a lower safety factor would be required in the assessment of first yield to ensure acceptable failure probability. The final line of the table shows the variation of required safety factor with ultimate strength ratio and allows immediate assessment of safety factors appropriate to ultimate strength ratios within the range 1.0 to 1.4.

5.5.2 Results of reliability analysis for other structures

Variations in the values the safety factor are observed for structures where the relationship between the green water load and the freeboard exceedance is different from the above. For the example of a vertical structure (the total load on a deck house, column or breakwater):

- The pressure will increase approximately linearly with the freeboard exceedance.
- The force (shear) on the structure will therefore increase quadratically with freeboard exceedance (until it is overtopped).
- The moment at the base one of these structures (assuming it is structurally a cantilever above the deck) will (until it is overtopped) increase with the cube of the freeboard exceedance.

Conversely, the loading on a horizontal pipe attached to the deck will be linear with freeboard exceedance. It is therefore necessary to consider a range of power terms from 1 to 3.

Figure 5-12 shows that this power relationship will have an important effect on the required safety factors. Interestingly, it also shows that the effect reverses according to whether the design is to 100 year, ULS, loads or 10000 year, ALS, loads. So an ALS design will require a smaller load factor for a cubic varying load than for a linearly varying load whereas a ULS design will require a higher safety factor for the cubic varying load. Though this is perhaps a surprising result, it is sensible because the ratio of the ALS design load to the ULS design load becomes progressively higher as the power relationship increases.

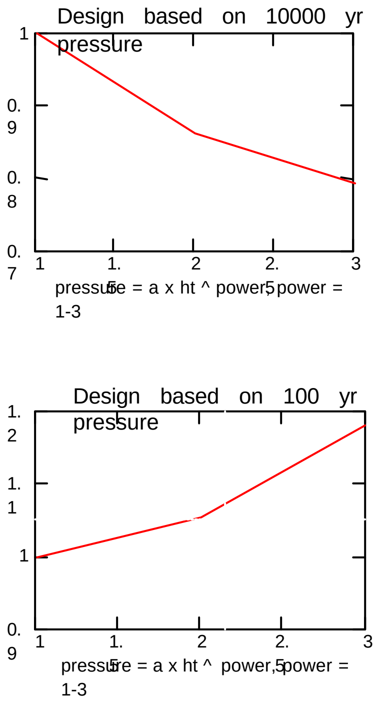


Figure 5-12 Variability of load factor with loading relationship

Notes:

1) Additional adjustment to the load factors will be required to allow for the different USR in shear and bending.
2) No overtopping is assumed.

Full calculations for the window and other typical deck structures can found in the reliability analyses detailed in the WP4 Technical Reports.

5.6 Structural design using Method 2

5.6.1 General

Different methods of structural design will be appropriate according to the importance of the structural component, the available resources and whether the design is preliminary, final or being reassessed. Detailed analysis may be appropriate for important structural items at the final design or reassessment stage but time limitations may mean that a simpler, but conservative analysis is often performed.

Detailed analysis is not appropriate at the preliminary stages of design because the effort involved is likely to be wasted as the structure changes during the design process. However inadequate assessment of loads during preliminary design can lead to severe problems later due to unexpected weight increases, additional material requirements or significant post fabrication contact variations. It is therefore necessary to provide a suitably conservative check at this stage.

5.6.2 Dynamic response

At a single location on a deck house structure, the apparent rise time for pressure loading is relatively short. This is caused by the rapid change in status from in-air to submerged as the surface of the water immerses this location. However, the total load on a substantial vertical part of the structure (e.g., stiffener, window, door) increases much more slowly, as it takes a finite time for the water height at the face of the structure to rise and load the entire structure. This effect is illustrated by Figure 5-13, below, which shows local pressure and total load on a typical deck house stiffener.

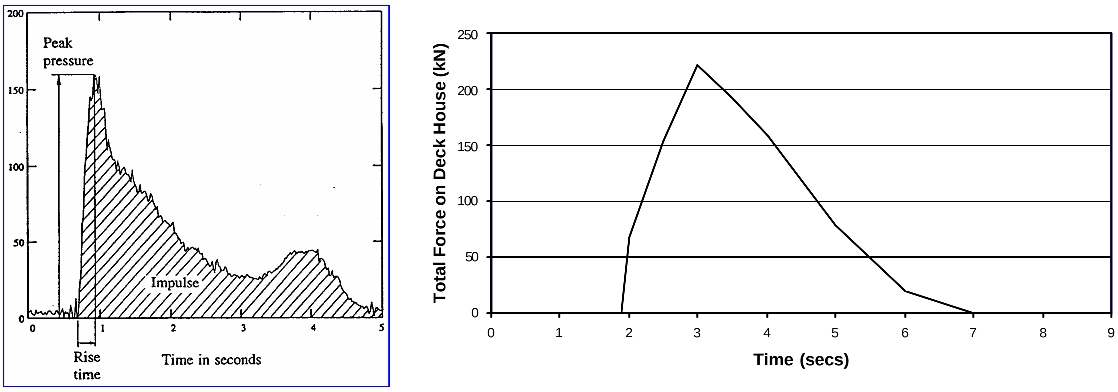


Figure 5-13: Pressure at a point and total load on deck house

If the rise times of the loading are sufficiently long, significantly greater than the natural period of the structural component being checked, then the dynamic amplification of this loading is small and static analysis is sufficient. In all the examples of deck structures considered within the SAFE-FLOW project (see Technical Report), dynamic amplification was found to be low, but not insignificant. This is illustrated by Figure 5-14, below, where the Dynamic Amplification Factor (DAF) is in the region of 1.2.

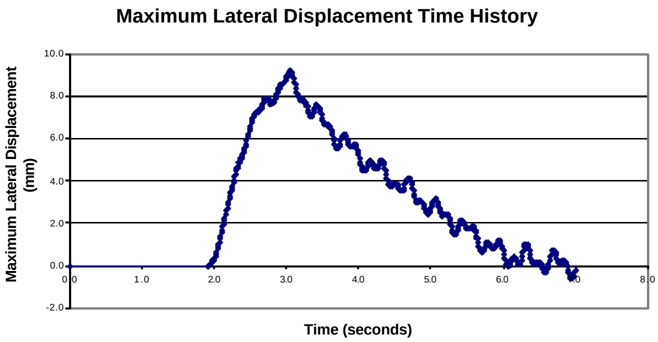


Figure 5-14: Response to loading in Figure 5-13

It is possible to treat the above dynamic effects by the simple introduction of a DAF. Suitable DAFs for varying rise times are given in Section 6 for wave impact. Similar values may be used here. Typical natural frequencies are given in the Technical Reports.

In other cases where natural periods could be longer, dynamic analysis or a quantitative assessment of dynamic amplification may be required. Higher amplification can also occur for horizontal structures, which can be loaded essentially simultaneously by green water. Structures in both categories are likely to include the following:

- Vertical cantilever structures, such as fire protection structures.
- Unbraced portal structures, such as helidecks.
- Pipework running horizontally across the deck.
- Horizontally stiffened deck houses.

5.6.3 Design by detailed analysis

The recommended approach for detailed structural analysis is to use time histories of calculated load (using for example the CFD package ComFlow) or of model test pressure measurements to load the structure. Typical pressure contours are illustrated by Figure 5-15, below, which shows pressure traces on the front face of a deck house at different times during a green water event. Suitable data is contained in the Technical Reports that may be scaled to suit the pressure magnitude and immersion height for a given green water event.

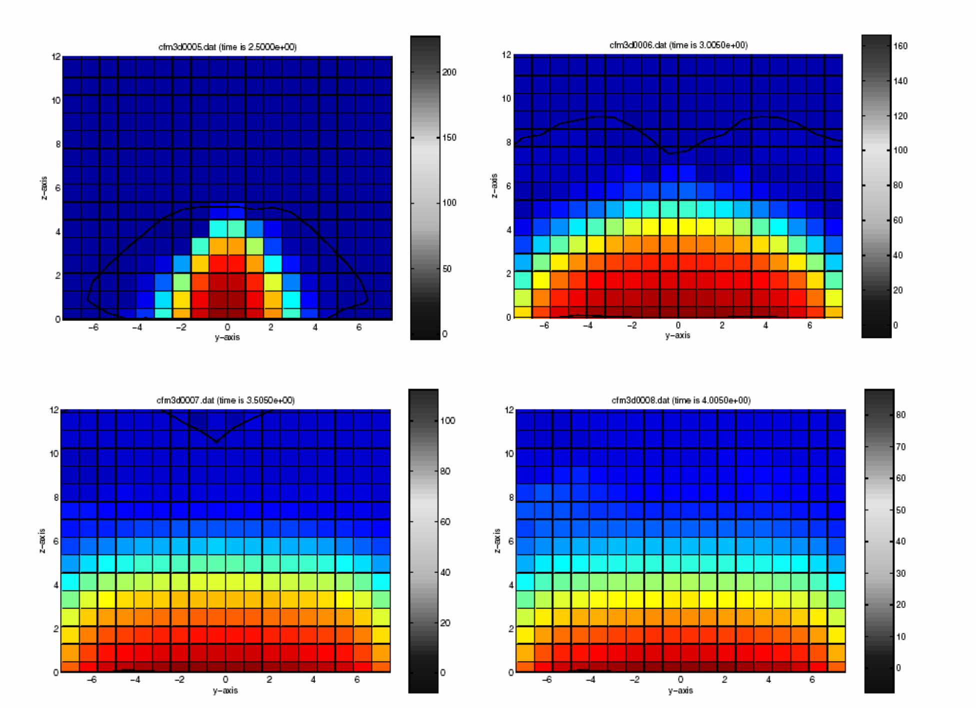


Figure 5-15: Typical deck house pressure contours

An appropriate Mean Load Factor should be applied, based on prior reliability analysis. Inelastic design requires the structure to remain essentially intact after the event with no rupture, although plastic response, limited buckling and yielding is permitted. It is recommended that strains in the structure should not be permitted to exceed 5%, to reduce the possibility of fracture. Other practical constraints on structural design are given in Section 3 of this report.

The type of structural analysis will depend on the nature of the structure:

- A simple vertical cantilever may be analysed on a spreadsheet, whereas a complicated structure may benefit from a detailed finite element analysis.
- The structure should be designed to behave in a ductile manner (with stocky sections, tough material and welds, care taken to avoid local severe stress concentrations caused by geometry or inadequate fillet weld sizes and avoidance of load paths through the thickness of steels with uncertain through thickness strengths). This will allow the designer to choose the way in which the structure resists the load and this may not be according to elastic theory. In this case there may be little point in performing a time consuming finite element analysis.
- An existing structure that cannot be shown to behave in a ductile way and that cannot, by simple calculations, be shown to be capable of resisting the design loads might benefit from a detailed finite element analysis. This might either:
    - o show that there are secondary load paths which reduce the peak internal forces to acceptable levels or
    - o identify the local areas that need to be strengthened.

Typical stress analysis of a deck house stiffener is illustrated by Figure 5-16. Examples of design by detailed analysis are given in the Technical Reports.

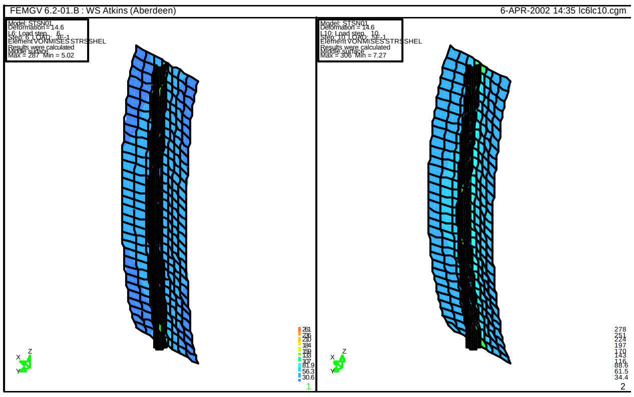


Figure 5-16: Transient dynamic analysis of a deck house stiffener

5.6.4 Design by equivalent design loading

Rather than perform complex transient dynamic analysis of the structure, it is preferable to perform much simpler design analysis with a uniform pressure load representing the true temporal and spatial variation of load. If this equivalent loading can be shown to be representative of the full transient load on the structure, then the structure may simply be analysed and designed as if it were subjected to a uniform static pressure of appropriate magnitude.

Examples of the derivation of equivalent design loads are presented in the Technical Reports. These loads should include the following:

- Dynamic effects, if significant, should be included as a dynamic amplification factor based on simple DAF calculations.

- Spatial and temporal effects should be considered by assessing the structure for the envelope of applied load.
- Mean load factors appropriate to the design conditions being undertaken for the structure.

A typical envelope of design pressures with height is given in Figure 5-17.

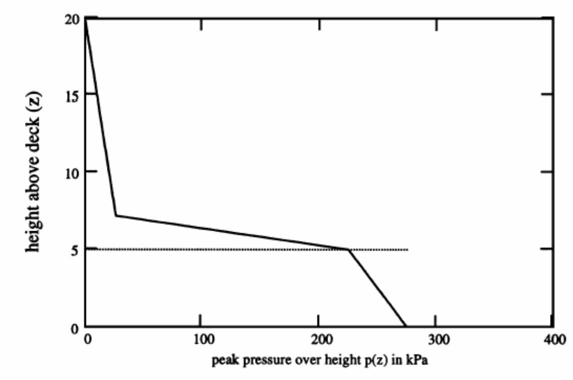


Figure 5-17 Typical envelope of pressure load

5.6.5 Equivalent elastic design

If the ALS condition governs, then the structure may be designed inelastically for the applied load. As an alternative to this complex analysis procedure, it is possible to design the structure for a reduced level of load termed the Equivalent Elastic Design Load (EEDL). This level of load is calculated by dividing the design load by an Ultimate Strength Ratio (USR).

Equivalent Elastic Design Load = (Equivalent Inelastic Design Load) / (USR)

USRs are set so that elastic design of the structure at the EEDL results in the same structural sizes as inelastic design at the original full design load.

There is no reason why transient load design should not be treated in the same way, dividing the temporal and spatial distributions of load in Section 5.6.3 by the USR to produce load time histories for linear finite element analysis. Stresses should be checked against yield, but in such cases, it may be necessary to accept local yielding where this is caused wholly by discontinuities in the structural model.

USRs vary with structural type and design. Typical values are presented in the Technical Reports and in Section 5.7, by structure type. These values have been based on analysis of different typical structural configurations. Where specific structural components are not covered by SAFE-FLOW, then the designer can derive specific values using similar methods.

In such analysis, suitable strain limits should be imposed on the structure to prevent fracture at extreme load. The structure should also be compact, else local buckling and tripping of flanges should be considered in the analysis. Specific considerations for different types of structure are presented in Section 5.7.

Highest USRs are produced for deck and deck house plate, since first yield of this structure type occurs in bending, yet the major load carrying capacity of the structure is produced by membrane action.

The USR of stiffener and frame structures depends very significantly on the extent of end fixity. Simply supported members are expected on deck houses. Failure of such structures will occur when a plastic hinge forms at mid-span, hence a USR of about 1.35 is obtained. If the stiffeners were continuous at their ends, then significantly higher values would be obtained.

5.7 Design guidance for different types of structure

5.7.1 General

For the purposes of green water loading, deck equipment can be classified as follows:

- Items of equipment or structures that are large enough to stop or significantly deflect the flow of green water. These structures include deck houses, turrets, separators, etc.
- Deck protection structures, such as breakwaters and louvered walls.
- The deck itself, which is subject to high pressure loads, particularly in front of deck houses.
- Items of equipment that are not large enough to significantly modify the flow, including walkways, flowlines, framed support structures, etc.

It is recognised that even small items of equipment can have an effect on water velocities, if these are sufficiently congested to block or funnel flow. Flow modification is not addressed directly by SAFE-FLOW but should be considered by the designer, where significant. Simple momentum calculations or the se of CFD numerical tools may be considered.

5.7.2 Design of deck houses and major equipment

Major items of equipment are classified as those that will have a significant effect on the flow of water over the deck, either diverting, arresting or even reversing its flow. In this category are deck houses, major items of process equipment, and significant support structures.

Deck houses include living quarters, access tunnels, blast walls and turret protection structures. These structures are often located at the bow or stern of the vessel, but in the case of turret structures, may be at midships. All such structures are potentially high enough and wide enough to prevent flow over a significant area, rather than simply diverting it.

The general arrangement of living quarters is characterised by a series of decks that are supported by the external stressed skin structure, which may be stiffened flat plate, swedged or corrugated. The stiffening in the walls normally runs vertically, acting also as columns to carry the vertical load from the quarters structure above.

The vertical orientation of such components is significant. It may be seen from the preceding sections that vertical structures are subjected to lower equivalent loads than critical horizontal members and also see slower total load rise times. Dynamic effects are therefore less pronounced. For these reasons, essentially vertical structures are strongly preferred.

Deck mounted turret structures often feature environmental shielding, turret bearings and mechanical gear of significant size that are likely to deflect flow of green water. The bearing structures are normally robust but include detailed equipment that may be damaged by green water. Protection structures are normally stiffened steel plate that could be at risk from water on the deck.

In most cases, production equipment is located on a process deck supported off the main deck by transverse frames. Unless these frame sizes become particularly large (allowing for potential fire protection around them), these structures may be treated as not affecting flow. However, some structures feature large transverse deck girders that effectively plate the area below the production deck. Except at openings in these girder webs, these structures will prevent through flow for at least the longitudinal component of green water velocity.

Steel structural components should be checked using codes of practice which are appropriate for the type of construction and that can take both axial and out of plane loading into account. Safety factors on the loading should be increased to account for the high variability and relatively large extreme values associated with green water.

Loads acting at the same time as the slam should be taken into account. These will usually include the weight of the structure and any live loading within it, both enhanced by vertical acceleration. In lieu of a detailed analysis, the significant acceleration could be used in this check.

Plated and stiffened deck houses follow similar construction to hull components, but are typically lighter and not detailed for fatigue. However, similar comments apply to those given for wave impact loading in Section 7.7. Natural frequencies and Ultimate Strength Ratios from that section apply. In calculating natural frequencies, the added mass of water acting on the panel should be considered. Suitable discussion is again given in Section 7. Deck columns are often simply supported, with the result that USRs would be lower than for continuous structures, in the region of 1.3.

Swedged construction is common. Where this is used, due consideration should be given to the variable stiffness of the plating in different directions, local buckling and the tendency for lateral pressures to cause local bending stresses in the plate that may be detrimental to its capacity. As is common with this type of construction, end connections are important. Shear integrity at the ends is particularly important, in light of the comments in Chapter 3, where it has been noted that there is less opportunity to redistribute shear forces than bending moments. There should be adequate capacity in shear. In general the USR is lower in shear than in bending so a higher safety factor will be required. This is particularly likely to dominate the design of the end connection details.

Doors and windows should be checked as structural items in their own right and the fixings, hinges and locks should also be designed to ensure they also have sufficient capacity. Note that small items such as doors and windows may need to be designed for higher average pressures than the larger structural panels around them. All details should be assessed for the nature of their failure under overload conditions to ensure that the overall behaviour is ductile. The mean load factor should be compatible with the USR of the components: materials lacking ductility (e.g., as a result of material properties or localised high stress areas) should be avoided where possible and if used then an appropriately higher mean load factor should be estimated from the reliability analysis.
Overhanging parapets on green water impacted walls have been found to cause high pressures when they redirect the flow. Structural components and windows etc near these features should be designed for the possibly increased pressures.

The overall fixing of very large deck house structures to the deck should be checked for adequate strength. In addition the local strength of stanchions or swedged plates at the deck connection should be checked to ensure that local bending moments do not lead to a failure that could run along the deck – structure connection. If the stanchions are designed to be encastré at the deck connection, then there must be sufficient reinforcement under the deck and the calculations should document the load path that can transfer the end moments into the hull structure.

5.7.3 Design of deck protection structures

Protection structures are often used to provide additional barriers against green water flow, as an alternative to raising freeboard elevations, strengthening deck structures or limiting operation and access to susceptible areas of the deck. Bulwarks are considered part of the external hull structure and have been included under wave impact loading. Two other principal means of providing protection are common:

- Breakwaters of various designs to prevent water flowing along the deck.
- Louvered walls to protect the sides of the vessel.

The structures may be represented as two types, those that prevent or deflect flow, and those that permit partial through flow. These are considered in the following sections.

Breakwaters have previously been classified into three main types:

- Forward staggered, v-shaped in plan.
- Vertical lower section with a forward staggered upper section, v-shaped in plan.
- Vane or louvered type, straight in plan.

The forward stagger of the breakwater helps to prevent green water flowing over the top. Vanes attract a smaller load than rigid breakwaters but this is consistent with the smaller angle through which the green water is deflected.
Current design rules produce robust structures that are supported by deep stiffeners from behind and cantilever upwards from the deck. However, it is difficult to provide suitable continuity below the orthogonally stiffened deck for V-shaped breakwaters and it is unfortunately likely that the full strength of these breakwaters cannot be mobilised.

This should be addressed in design/reassessment by a first principles analysis of the load paths. It should also be addressed in any future code or class rule recommendations for the design and assessment of these structures.

Breakwaters (and some equipment) may be designed to be overtopped in extreme conditions. Figure 5-18 shows that typically, on overtopping, the pressures reduce before increasing again. If the breakwater is not expected to be overtopped then the design mean load factors should be based on a quadratic force and a cubic moment – bow exceedance relationship.

If overtopping is likely, then mean load factors for a linearly varying force and quadratic moment – bow exceedance relationship should be selected, but the design load should not be less than the overtopping load with the appropriate safety factor.

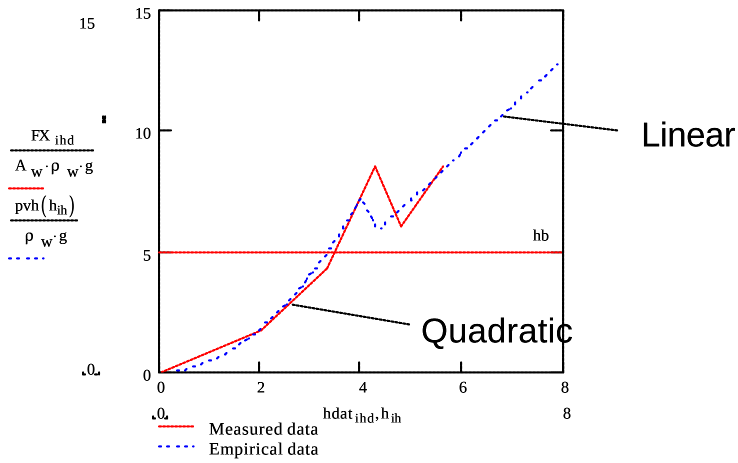


Figure 5-18 Breakwater pressure versus depth of bow exceedance

Side protection structures tend to be louvered to deflect flow away from the deck whilst permitting any accidentally released gases to escape. These could be designed to cantilever up from the main deck or to span between the main deck and a higher working deck. The latter is preferable because it should result in a more efficient structure and a higher USR through the formation of three plastic hinges before failure.

Similar structures have been considered for the bow. In such cases, the upper structure may be supported by cantilevers or props, but it is particularly important that the connection of these structures into the deck should be adequately designed and suitable load paths provided.

Sharp corners on structural members should be avoided and the design and the implementation of welds and local details of any fairings welded to the supporting structure should be reviewed to ensure that they do not reduce the ductility. The higher shear mean load factors should be applied to the shear forces for end connection design and the connections should be at least as strong as the members to maintain the ductile behaviour.

5.7.4 Pressure loads on deck plate

Significant pressures can be imposed on deck plate due to green water flow. Based on data from the Green water JIP, these pressures are not a particular concern away from congested areas. However, adjacent to deck houses and other structures that deflect the flow, substantial increases in pressure are caused when the water impinges on the target structure. This effect produces similar magnitude and time history of pressure acting down on the deck as on the deck house. Design of deck structure plate, stiffeners and girders in the vicinity of deck structures should include this effect. Design guidance is expected to be very similar to design of the deck house structures themselves.

5.7.5 Loading on minor items of equipment

In this classification are items too slender to substantially disturb the flow. Loading calculations for these items will be based on the incident fluid velocity. Items of deck equipment and structure that may be classed as slender for these purposes are as follows:

- Small items of process and other equipment.
- Pipework spans and sizes.
- Safety equipment, lifeboats, liferafts, davits, etc.
- Personnel access, walkways, stairwells, etc.
- Winches, bollards, etc.
- Electrical cables, lighting, etc.
- Structural sizes and connection details into the deck for the following:
    - flare tower legs,
    - helideck legs,
    - pallet, PAU or piperack supports.

Any small diameter structures encountered by this flow will be subject to an initial slam force given by:

$F = ½ C_s D L \rho V^2$

where:

C_s = a slam coefficient
D = the member's diameter or width
L = the member's length
ρ = the water density
V = the water particle velocity for the flow over the deck

The velocity should be the incident deck flow velocity, increased if blockage or funnelling effects are likely.
Then a drag force on the structure will occur, given by:

$F = ½ C_d D L \rho V^2$

where: C_d is the drag coefficient selected according to the shape of the structure/equipment.

In addition, a buoyancy/lift force may be important for horizontal members.

As for the breakwater, vertical prismatic structures cantilevering upwards from the deck will typically have mean load factors based on a quadratic force and a cubic moment. Figure 5-12 indicates the effect of the linear to cubic nature of the load on the mean load factor. Overtopping of slender structures may be possible. The design load should not be less than the overtopping load with the appropriate safety factor.

Again, particular care is required with end connections that should be designed for minimal yielding and should either be designed with the increased shear force mean load factors, or to be as strong as the structure they are attaching, whichever gives the larger weld.

6 WAVE IMPACT

6.1 Introduction

This Chapter considers extreme wave impact type loading on the external wetted surface of the vessel, including bulwarks. This section deals with a description of the characteristics and probability of bow impact loading, an understanding of which is considered to be essential prior to the presentation of the Structural Design Methodology in Chapter 7.
Sources of wave impact load data used in this report are briefly listed in Section 6.2. Several of these sources of data are presented as Annexes to this methodology, which should be referred to for more details.

It is clear from the data that there are various types of wave impact 'event' that may be identified, from short sharp impacts to more progressive and extensive pressures over the face of the structure. The spatial extent and rate of loading is important since it causes different static and dynamic response in the structure. Data from full-scale monitoring and from tank tests is summarised in Sections 6.3 to 6.5.

Section 6.6 finally discusses the dynamic response to wave impact loads. The dynamic response of the structure requires possible dynamic amplification to be included in design calculations.

6.2 Sources of data

The following main sources of data were available during the SAFE-FLOW project for the derivation of loading due to wave impact:

- Design rules and guidance, historical and experimental data in the public domain.
- Tank tests and studies carried out by Glasgow University and interpreted by WS Atkins as a result of bow damage to the BP Schiehallion FPSO in November 1999. These tests were on a fixed, instrumented bow component subject to very steep incident waves.
- The results of a bow monitoring system installed on the Schiehallion vessel in January 2000, giving approximately 3 years of recordings. Note, however, that correlated wave data was not available during this time due to difficulties with the placement of a planned wave rider buoy. More details can be found in the WP3 Technical Reports.
- Limited bow impact pressure data, mostly on flare slamming, obtained as a result of the JIP 'FPSO Green water loading' carried out by MARIN. This data principally covers flare slamming on bows of high rake angle.
- Tank tests carried out by MARIN under Phase 1 of the SAFE-FLOW JIP during 2001. Irregular sea incident wave data, vessel motions, bow pressures and structural response results are available on a floating model of the Schiehallion FPSO.
- Tank tests carried out by MARIN under Phase 2 of the SAFE-FLOW JIP during 2002. Irregular sea incident wave data and bow pressure results are available on a fixed schematic bow structure with varying rake and plan angles.

- Tank tests being carried out by Glasgow University in 2001 as part of EPSRC/HSE research. The instrumented Schiehallion bow was incorporated into a free-floating hull model to include vessel-wave interaction, fore-foot slamming and hull girder bending due to slap and slam on the bow. A further model of the Loch Rannoch shuttle tanker investigated an alternative ellipsoidal bow form and varying wave directions.

6.3 Information from full-scale monitoring

6.3.1 System description

Full-scale measurements on the bow of the Schiehallion FPSO were reviewed as part of the SAFE-FLOW JIP. This gives insight into impact loading and structural response during real ocean events. Relevant details of the monitoring system are recorded below and illustrated in the figure that follows:

- The system was installed in Dec 99 / Jan 00, comprising:
  - o 16 pressure sensors;
  - o 26 strain gauges;
  - o 2 accelerometers;
- Data collection commenced 19th Jan 2000.
- Statistical data was continuously recorded.
- Pressure sensors trigger detailed data collection at 3 bar (30 m pressure head).
- Detailed impacts up until the end of January 2003 have been reviewed.

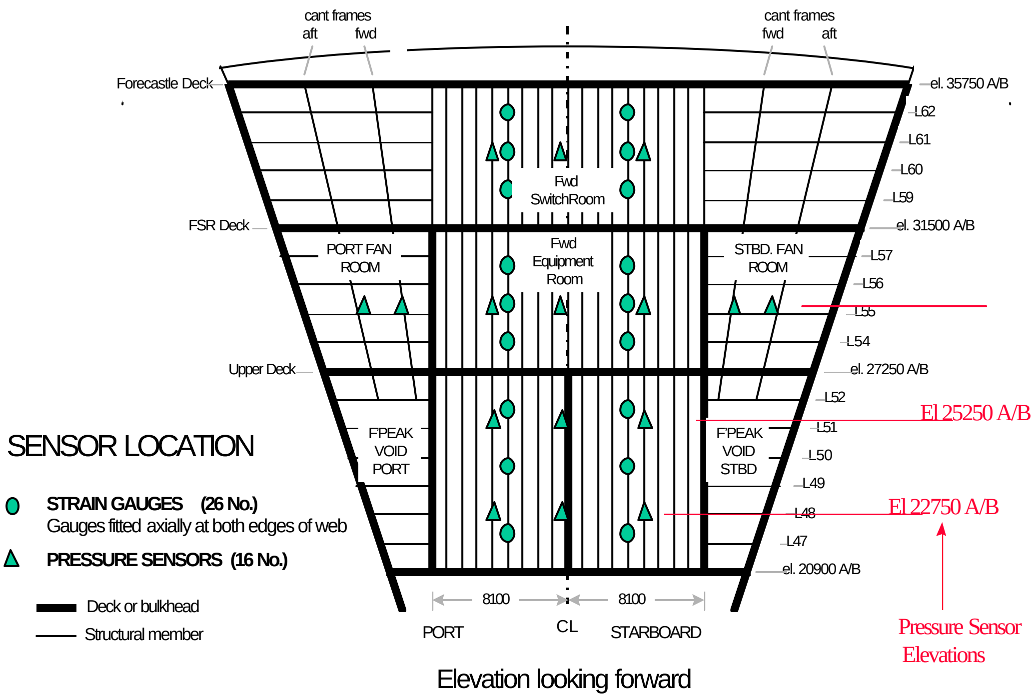


Figure 6-1: Schiehallion full-scale measurement set-up

6.3.2 Key results obtained

Key results from the first three years of operation were as follows:

- A total of 82 detailed data sets collected over first year (of which 14 were spurious).
- High pressures were generally recorded at only one sensor for each event, with a few significant exceptions.
- No impact events resulted in reported damage to the strengthened structure.
- The maximum local pressure was 134 m head on 22nd Nov 2002.
- The maximum reliable stress ranges were in the region of 350 MPa.

No wave monitoring device was available during operation of the above system, so that pressures could not be correlated with wave data.

6.3.3 Observations from individual impact events

The following observations were made from individual impact (pressure and stress) records reviewed:

- Almost all (91% of) peak events occurred below the upper deck, indicating that the restricted operating draught is successfully limiting the height of impact loading.
- Despite this predominance of records low in the structure, peak loads on the lowest three rows of sensors did not reduce that rapidly with height (134, 96 and 91 m head). This is expected to be due to the greater height of waves that impact higher gauges, offsetting the lower frequency of these events.
- There was a predominance of high pressures on the port side, which may be related to vessel operating practice and prevailing weather.
- There was a significant yearly variation in impact intensities, with the winter of 2002/3 being particularly severe and the winter of 2000/1 being very calm.
- Five main types of impact pressure trace were identified:
  - 41% are short *Sharp* slams
  - 32% are *Traditional* shaped slam events
  - 14% occur while sensor is *Immersed*
  - 9% of slams are *Dual* impact events
  - 4% have *Broad* peak pressures

More than one type of slam pressure record can occur during any one event, as illustrated by the following typical impact:

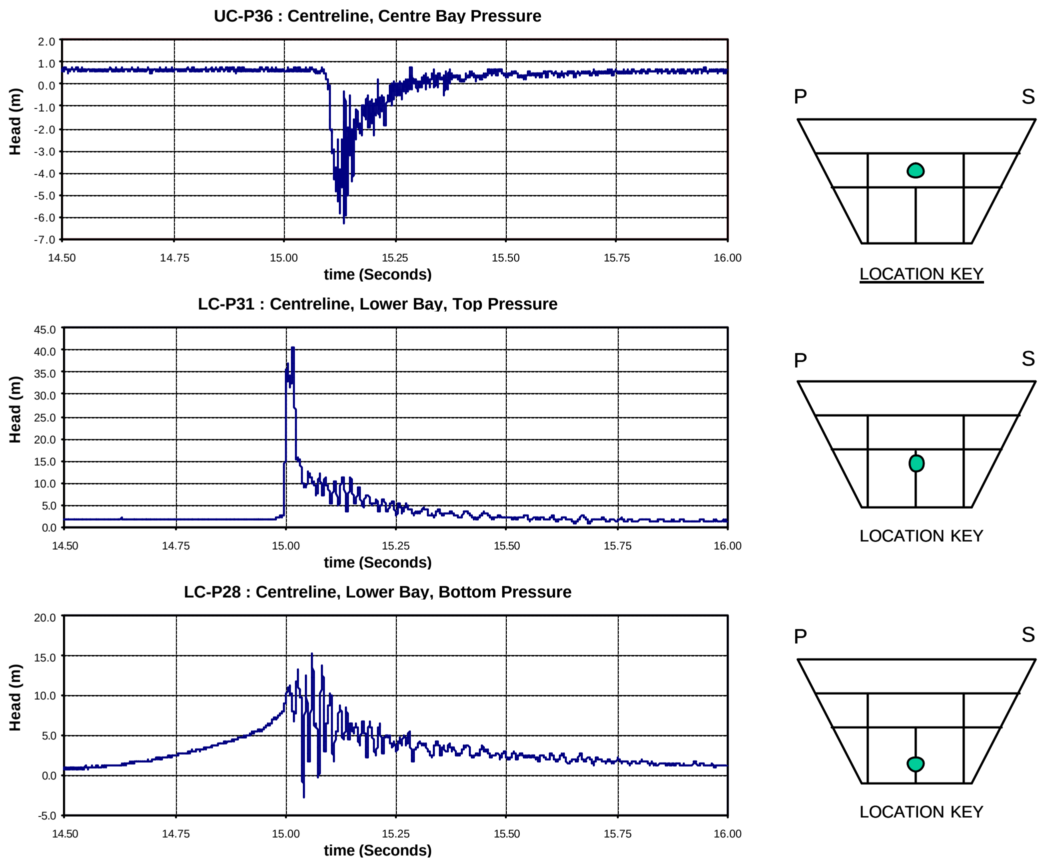
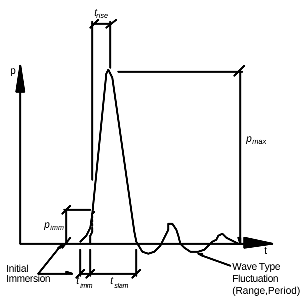
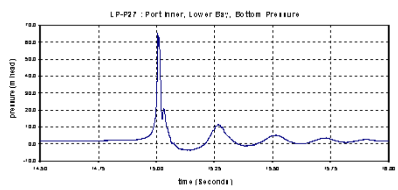


Figure 6-2: Typical slam record

- The most frequent and significant ‘sharp’ events are characterised as follows:

Sharp events have both rapid rise and decay times



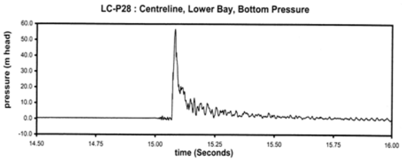


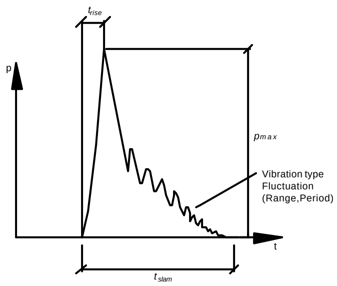
- Rise times typically rapid, normally 0.01 seconds
- Total impact duration short, less than 0.1 seconds
- Maximum recorded pressure head is 134 m
- 36 events

Figure 6-3: Sharp slam event

- Traditional slam events are characterised as follows:

Traditional Impacts have a rapid rise time and exponential decay





- Rise times typically 0.01 to 0.05 seconds
- Total impact duration from 0.10 to 0.50 seconds
- Maximum pressure head of 100 m
- 28 events

Figure 6-4: Traditional slam event

- Most impact events may be classed as localised, in that they do not cause high impact pressures (>30 m head) at more than a single sensor.
- The limited spatial extent of these impacts makes it difficult for individual sensors to reliably record the maximum impact event. The sensors may simply not be located at the correct position to pick up peak pressures.
- Once an impact event occurs, oscillations are observed in the pressure traces. Some of these, but not all, correspond to stiffener natural frequencies. These effects are most pronounced for ‘immersed’ and ‘dual’ impacts, where significant pressure fluctuations result at immersed gauges. Several of these immersed gauges show pressure fluctuations at 60-100 Hz, which does not correspond to fundamental stiffener frequencies, but which may be higher modes, or plate vibration.
- A particularly pronounced ‘wave’ type of pressure fluctuation is frequent (particularly for sharp impacts). Its frequency of 4-10 Hz does not appear to correspond to any likely structural period. This fluctuation may be caused by pressure waves in the water which is made compressible by entrained air bubbles and pockets.

Traditional slams, which result in a ‘peak’ stress record, showed a good correlation between the magnitude of the pressures and stresses.

The correlation between pressure and stress is not as good for short sharp events. This may be due to: (i) the more localised nature of these loads; (ii) more significant but variable dynamic response; (iii) variable response depending on where the impact occurs on the stiffener; and (iv) how close a pressure sensor was located to the location of the peak impact pressure.

No single feature of any impact event dominates the structural response produced in the bow stiffeners of Schiehallion, but a combination of identifiable impact parameters can give an indication of the resultant stress magnitude:

- Magnitude – not that firm a relationship
- Slam Elevation – higher impacts cause higher stresses
- Rise/Fall Times – important for short sharp slams
- Immersion Speed – rapid slams tend to be severe
- Spatial Extent – extensive impacts typically worse
- Structure Location – relative to slam elevation
- Bow shape – local curvature and ellipticity
- Dynamic Response – sharp slams increase dynamics

Statistical data in the form of the number of pressures in 5 m head 'bins' was produced by a rainflow counting method. When combined with records of individual events, the probability of different slam pressures over the monitoring period could be produced.

The following figure shows the average number of 'events' of different magnitude likely to occur each year:

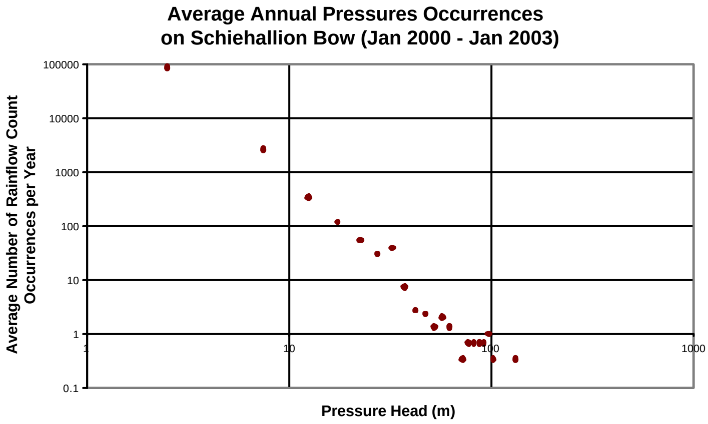


Figure 6-5: Average annual pressure occurrences at Schiehallion

It is apparent from this figure that very large local maximum pressures may be expected through the life of the vessel. However, it was also shown that these local high pressures were not particularly representative of the average load along a stiffener, this average load being significantly less, particularly for sharp slam events.

6.4 Summary of MARIN model tests

As part of the SAFE-FLOW project MARIN performed 2 series of model tests:

- Model tests on free floating Schiehallion FPSO in Phase 1.
- Model tests on a highly instrumented fixed simplified bow in Phase 2.

The tests on free floating Schiehallion FPSO in Phase 1 were focussing on the motions of the FPSO, the local relative wave motions at the bow and the local pressures and sectional loads at the bow.

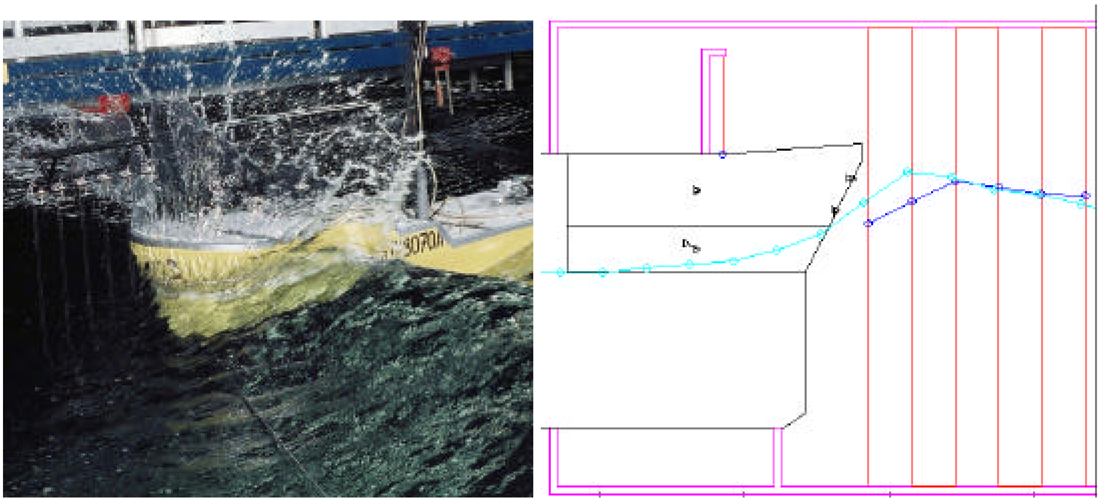


Figure 6-6: Bow impact event on free floating Schiehallion model and visualisation of wave fronts

The tests showed that more detailed load measurements were necessary and that an investigation was needed into the relation between the incoming waves and these loads. This resulted in the model tests on a highly instrumented fixed simplified bow in Phase 2.

The bow was instrumented with a large array of pressure transducers and 3 force panels. The test program, also making use of extensive video recordings, was designed such that it was possible to determine the correlation between undisturbed wave shape and the impact pressure time traces.

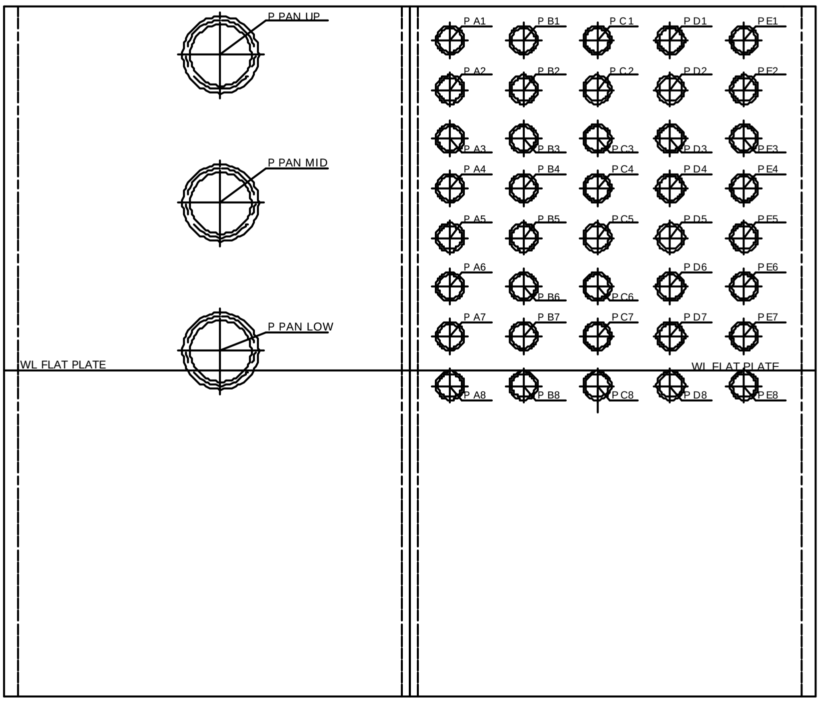


Figure 6-7: Schematic bow model in Phase 2 tests with array of pressure cells (right) and 3 force panels (left)

It was found that wave impact at the bow is highly correlated with the local wave steepness. More details on the outcome of these tests will be presented in the next sections. Figure 6-8 shows the typical stages during a bow impact:

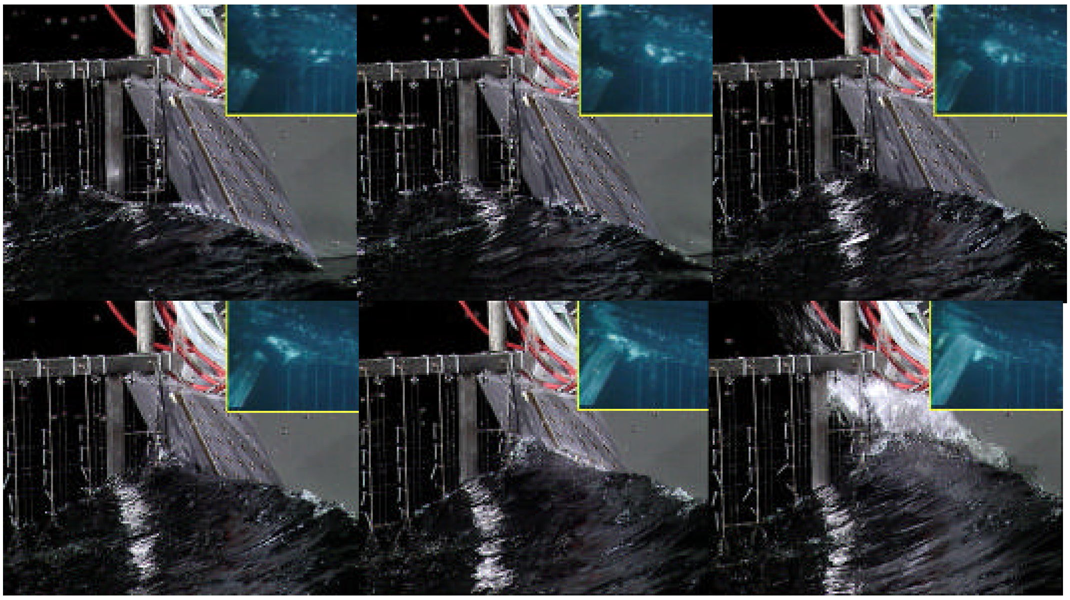


Figure 6-8: Typical stages during a bow impact

6.5 Summary of EPSRC model tests

The Naval Architecture Department at Glasgow University (now Naval Architecture & Marine Engineering of Glasgow & Strathclyde Universities, NAME) has been working in parallel on the subject of impulsive loading on FPSOs in extreme seas based on an EPSRC-Link grant with additional funds provided by BP, HSE and an IMarEST Scholarship. This enabled tests on the Schiehallion FPSO and the Loch Rannoch shuttle tanker. The FPSO models were built at a scale of 1:80 and were segmented along their length (to allow the measurement of hull girder bending) and in two or three parts in the bow (to allow the measurement of overall bow impact forces, or more accurately the bow response to bow impact loads). They also had arrays of pressure transducers built into the bow for local pressure measurement.

Figure 6-9: Loch Rannoch model

Focussed wave groups were used in the testing. Both High and steep and intermediate waves were used, see Figure 6-10:

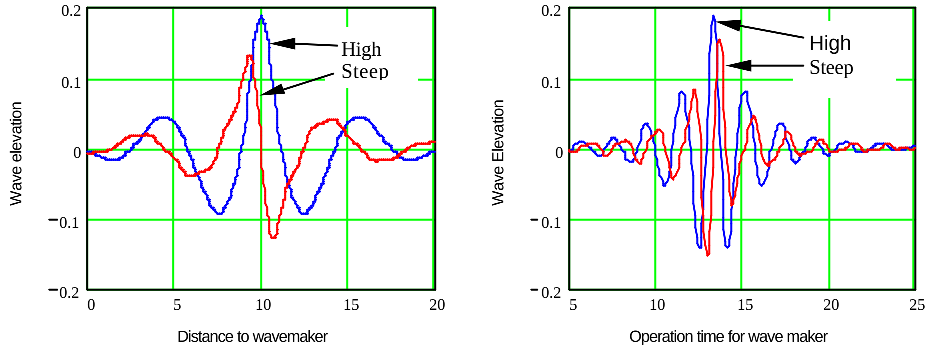


Figure 6-10: Average shape of highest and steepest waves expected once in 3 hours (model scale)

The notionally steepest wave group will not always give the highest slap pressures because it may well have broken before the impact, so reducing the slap pressures. Figure 6-11 shows a theoretical and measured steep wave time history which show signs of the crest before the main crest breaking.

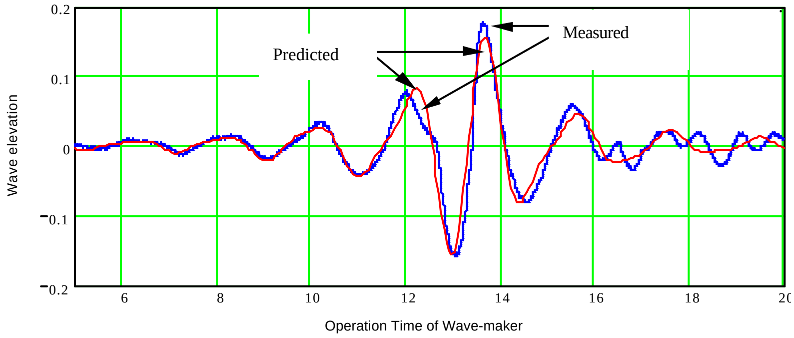


Figure 6-11: Comparison of predicted and measured steep wave group time history (note breaking in crest prior to peak)

Figure 6-12 shows a typical resulting wave impact in a high steep fronted wave.

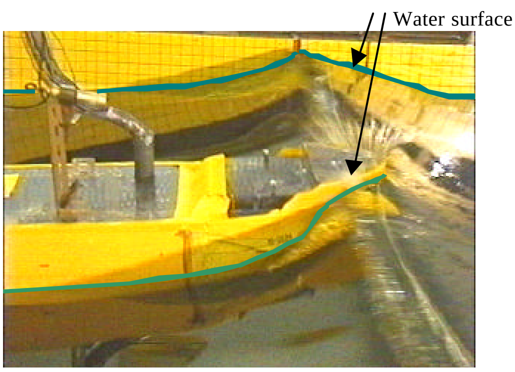


Figure 6-12: Typical resulting wave impact in a high steep fronted wave

The main outcome of the tests was:

1. Measurement of dynamic response and estimates of forces on the whole bow and on a variety of areas.
2. An understanding of the different types of wave that would give the worst impacts in different steep sea-states.
3. A relationship between the extreme events and the 'underlying' linear theory.
4. A pressure - loaded area relationship.

More details can be found in Barltrop and Xu (2004) and in section 7.5.2.

6.6 Wave impact prediction methodology

6.6.1 General

To calculate the probability of wave impacts on any given vessel structure, at any given elevation, a prediction method for the simulation of the water surface and the occurrence of wave impacts was created in SAFE-FLOW. To allow this simulation to accurately predict wave impacts, it was necessary to consider the following:

- What are the most significant wave parameters that can be used to determine when a slam will occur and what magnitude it will be?
- How can the probability of occurrence of these parameters be predicted numerically with sufficient accuracy to represent real seas?
- What are the temporal and spatial variations of pressure on the hull during the resultant impact event?

Evaluating the available results from all model tests performed within SAFE-FLOW, it was decided to develop a method that split up the problem in two main problems, see Figure 6-13.

- The relation between the local wave characteristics and the magnitude (and other characteristics) of the wave impact, the lower line in the figure.
- The position of the impact based on the related ship motions, the upper line in the figure.

The combination results in a localised impact with specific properties, resulting in the structural response of a local structure with its specific structural properties.

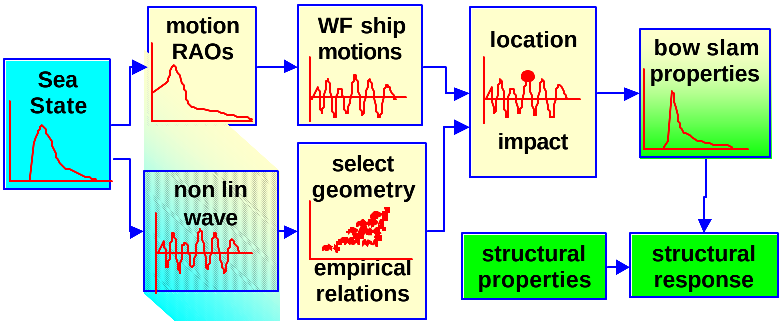


Figure 6-13: Load-response prediction methodology

Below the key aspects of the methodology will be presented and discussed. Design guidance and the step by step application of this methodology in the BowLab program will be discussed in Chapter 7.

6.6.2 Non-linear wave characteristics and the relation with slam probability

In the SAFE-FLOW project it was found that the magnitude of the wave impacts at the front of the bow are dominated by the wave characteristics (namely the local wave steepness), rather than by the motions of the ship relative to the waves (relative wave motions). The kinematics in steep fronted waves are such that the water particles move almost perpendicular to the bow, resulting in strong impacts.

This implies that the total problem can be split into an impact magnitude problem which is wave dominated and an impact position problem, which is affected by the ship motions. For other problems, such as bow flare slamming, this separation cannot be made, because the relative wave-hull motions drives the impact magnitude there (see Chapter 7).

The local wave steepness (d?/dx) can be determined from measurements of the wave elevations in an array of probes. An example is shown in Figure 6-14, which shows the spatial wave profile for successive steps in time. The time step between the different lines is 0.31 seconds and the distance between the probes 6 metre allowing for an accurate derivation of the local wave steepness.

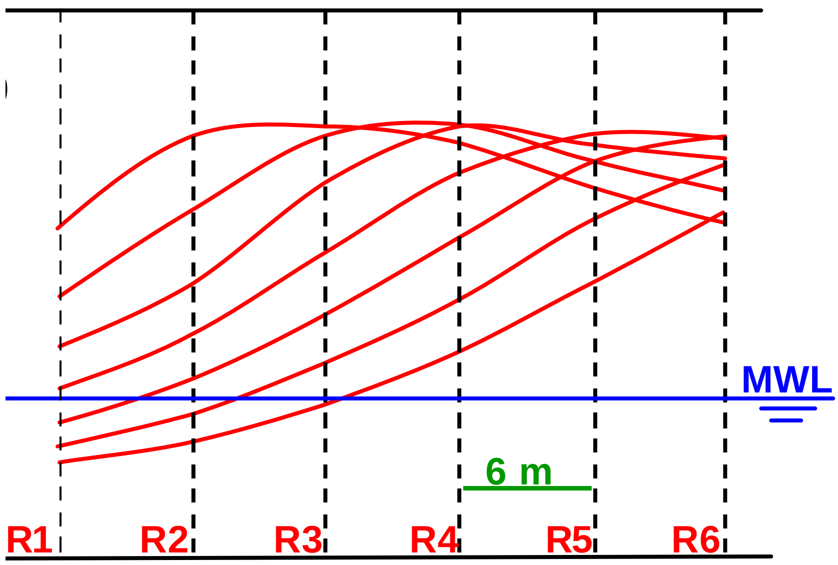


Figure 6-14: Visualisation of the local wave steepness (d?/dx) based on the measurements of the wave elevations in an array of probes

The combined spatial and temporal information of the sea state needed to derive the local wave steepness is not generally available (in full-scale data and model tests). Therefore the vertical free surface velocity is preferred as input to the model.

The vertical free surface velocity (d?/dt) is related to the local wave steepness (d?/dx) by the wave celerity, see Figure 6-15:

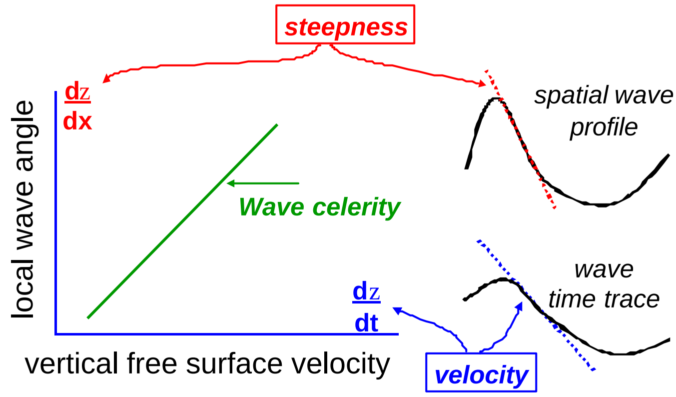


Figure 6-15: The vertical free surface velocity (d?/dt) is related to the local wave steepness (d?/dx) by the wave celerity

The relationship between the maxima in the vertical free surface velocity (d?/dt) and the impacts is shown in Figure 6-16. It shows the traced impacts (circles) versus the time traces of the vertical surface velocity. The impacts occur at the same moment as the maxima in the vertical free surface velocity.

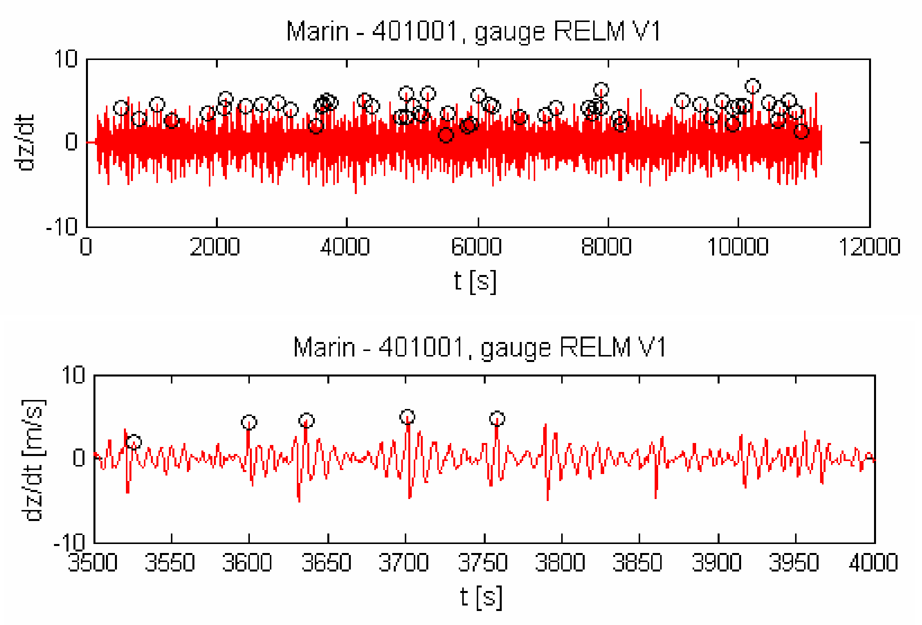


Figure 6-16: The traced impacts (circles) versus the time traces of the vertical free surface velocity

Another interesting observation is that the maxima in the vertical free surface velocity occur in the largest wave crests. The red (+) points in Figure 6-17 indicate the combinations of crest height and vertical free surface velocity for which impact did occur, for the black dots no impact occurred.

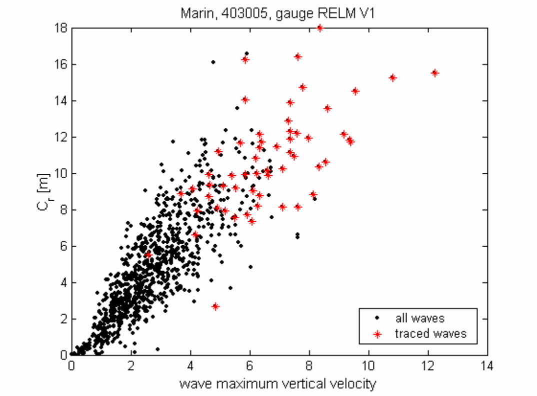


Figure 6-17: Crest height as function of vertical free surface velocity

It is clear that the impacts occur for large vertical free surface velocities.
The most suitable method of simulating the water surface to give a reasonable probability of vertical free surface velocity was found to be second order wave theory, as described by Sharma and Dean (1981) for instance. More details can be found in the WP1 and WP2 Technical Reports. In steep waves that cause the bow impact, linear theory clearly under predicts the wave steepness. Applying second order wave theory results in an improved prediction of d?/dt, as shown in Figures 6-18 and 6-19 for the basin waves applied.

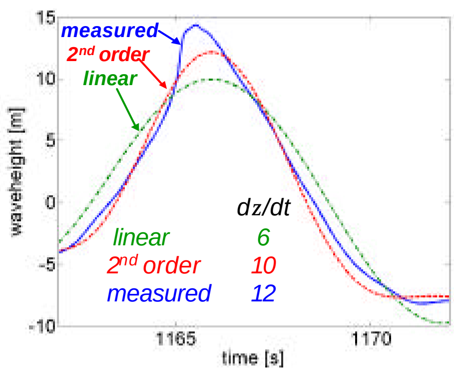


Figure 6-18: Measured, first order and second order wave time trace (from basin tests)

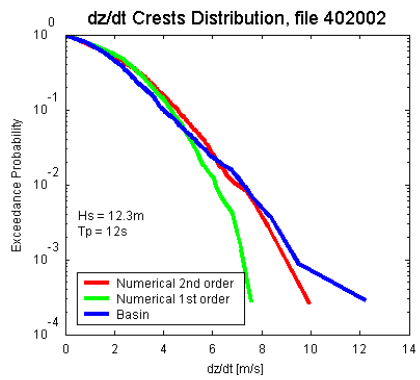


Figure 6-19: Probabilities of exceedance of vertical free surface velocity based on measurements in basin, first order theory and second order theory

A comparison was also with field data from North Cormorant collected during a storm in 11th-12th March 1996, see Figure 6-20. Again the second order model gave an important improvement in the prediction of the extreme vertical free surface velocities. More details can be found in the WP1 Technical Report.

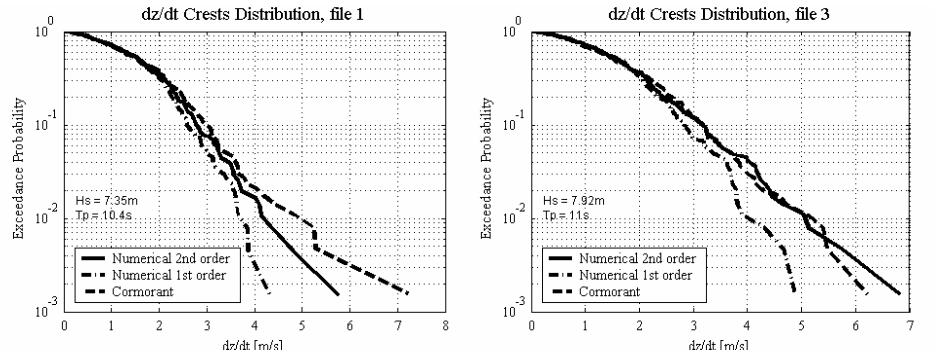


Figure 6-20: Comparisons of the cumulative probability of exceedance of vertical free surface velocity of field data and numerical simulation

To estimate the probability of an impact at a certain vertical free surface velocity, the percentage of impacts occurring were counted for bins of vertical free surface velocities. An example is given in Figure 6-21, see for more details the WP1 Technical Report.

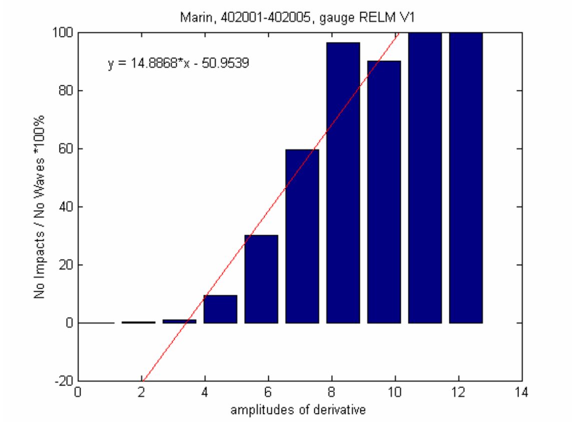


Figure 6-21: Probability of impact versus vertical free surface velocity

It is observed that up to a certain vertical free surface velocity no impacts occur. Above this threshold level, the probability increases linearly with the vertical free surface velocity up to the probability of 100%.

For the linear increasing part the probability of a slam occurring at a given value of d?/dt can be expressed as follows:

$$P\left(\text{Impact}\left|\frac{d\zeta}{dt}\right.\right) = 0.19633 \cdot \left(\frac{d\zeta}{dt} - 3.631\right)$$

For values of $d\zeta/dt$ below 3.631 no slams occur (P = 0) and for $d\zeta/dt$ above 8.724 a slam will always occur (P = 1).

6.6.3 Non-linear wave characteristics and the relation with slam magnitude

Beside the slam probability, the slam magnitude is of vital importance. After analysis of all data, it was decided to relate the slam impulse (I), the area under the load time trace, to vertical free surface velocity (d?/dt).

Figure 6-22 shows the measured impulses versus the corresponding vertical free surface velocities. For different velocity bins the mean and standard deviation of the occurring impulses is added to the figure, resulting in straight lines.

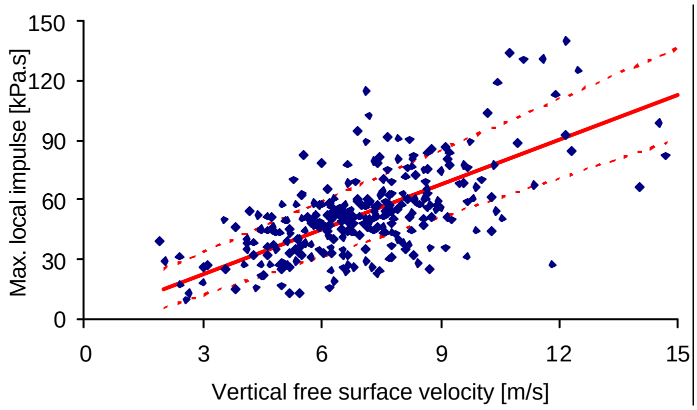


Figure 6-22: The measured impulses versus the corresponding vertical free surface velocities

The relation is independent of the sea state and holds for a schematic flat plate bow. Within the design method the mean fit is used as a maximum that can occur. For more realistic curved bow shapes the loads are reduced as described in the next chapter. The spreading around this mean can be used as input to the derivation of the load factors in a first principles reliability approach.
From this mean curve the maximum local impulse (I) can be calculated as follows:

$$I = \int p \cdot dt = c_I \cdot \frac{d\zeta}{dt} \quad \text{with} \quad c_I = 7.6 \frac{\text{tonne}}{m^2}$$

in which p is the local pressure and ? the wave elevation.

Other wave impact characteristics, such as rise time, decay time, spatial extent and the effect of the bow shape are later applied to this local impulse on a flat plate to determine the resulting structural response.

6.6.4 Position of the impact related to crest height

In the fixed bow model an array of pressure transducers was installed. In total 5 columns each of 8 sensors were available for the analysis. From the repeated wave tests it was found that large spreading can occur on the time trace of the pressure in exactly reproduced waves. Therefore the impulse of the slam is calculated with an integration of the pressure over its duration. The magnitude of the impulses is plotted for a column of transducers, see Figure 6-23.

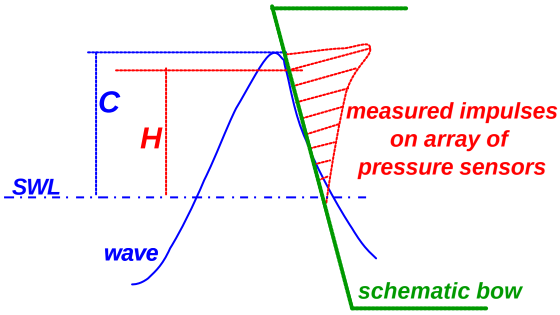


Figure 6-23: Definition of slam centre (H) with respect to crest height (C) of incoming undisturbed wave

From this column of measured impulses the slam centre (H) can be defined as the level above the free surface with the maximum impulse. This height can be compared with the crest height of the corresponding incoming wave (C). Figure 6-24 shows this comparison for the crest heights of the undisturbed waves compare to the slam centre.

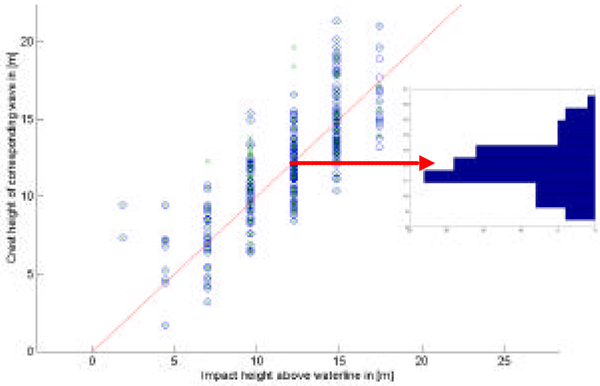


Figure 6-24: Slam centre (H) on horizontal axis as function of crest height (C)

Due to the adopted method the impact height above the waterline is sampled with the distance between the rows of the transducers. For each impact with the slam centre at the third row from the top a histogram of corresponding crest heights is shown in a small inset to the graph. It is clear that some spreading occurs but that most of the crest heights do correspond to the slam centre. The comparison with the wave heights of the incoming undisturbed waves show that most slam loads occur relatively close to the wave crest. This is consistent with the observation that the maximum velocity occurs close to the crest front of the incoming wave.

6.6.5 Position of impact related to ship motions

The previous section showed that the slam centre occur close to the crest of the undisturbed non-linear wave. However, for the design of a bow panel in a floating structure, the ship motions should also be taken into account. The ship motions in steep waves can still be described with linear diffraction theory. This was demonstrated by analysing the floating Schiehallion model tests results from the Phase 1 work as described on the next page.

An iterative procedure was developed that fitted a linear wave and its second-order contributions to a measured wave train. The procedure works as follows:

1. An estimate of the linear part of the wave train is given. In practice, a good first guess is the measured wave train itself.
2. With a Discrete Fourier Transform (DFT) the amplitudes and phases of the linear estimate are determined.
3. The corresponding second-order sum- and difference frequency wave is determined and added to give an estimate of the total wave.
4. The difference between the estimated wave and the measured wave is determined. If this difference is larger than a certain pre-defined value, the difference is added to the linear estimate and the scheme is repeated from point 2 onwards.

More details can be found in the WP2 Technical Report. After the scheme has converged, the linear amplitudes ς_i are known and the response to this linear wave can be calculated. The table below shows a comparison of the standard deviations from the calculation and the measurements for a head seas condition.

	Measurement	Calculation with linear theory
Wave [m]	3.63	3.62
Surge [m]	1.00	0.96
Heave [m]	1.50	1.32
Pitch [deg]	2.26	2.04

These statistics show that the motions are predicted reasonably well. The measured and calculated time traces for the wave, surge, heave and pitch are shown in Figure 6-25.

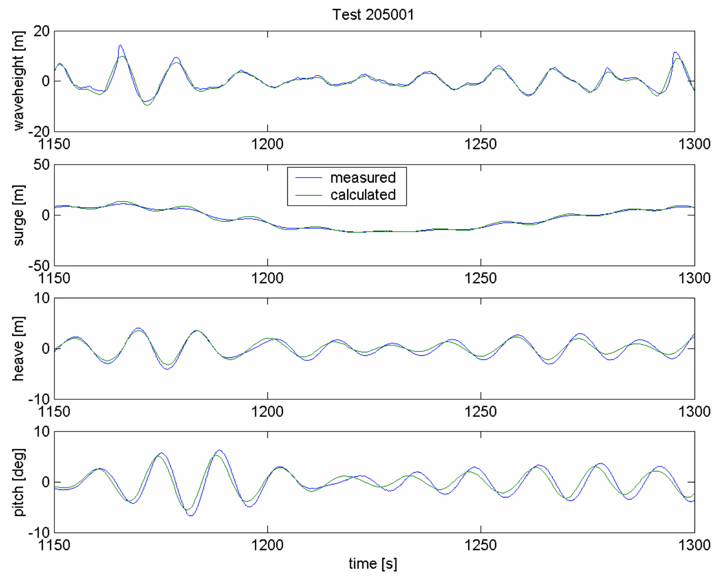


Figure 6-25: Time traces of measured vessel response compared to calculated response assume linear theory

Even for these large differences between the linear wave and the measured wave, the underlying linear wave defines the ship response with good accuracy.

6.6.6 Slam characteristics

From the comparison of the measured pressures and stresses in the Schiehallion bow monitoring it was concluded that the pressure duration, the immersion velocity and the spatial extent are important parameters to determine the load-response of a panel in the bow. The large number of pressure sensors and accurate wave measurements on the schematic bow model do provide this information. The present section discusses how these parameters were derived.
The pressure duration can be derived from the local slam time trace. Figure 6-26 shows a typical example of a slam that consists of a linear rise and an exponential decay. It is clear that this slam is completely defined by the rise time, decay half time and the impulse of the slam.

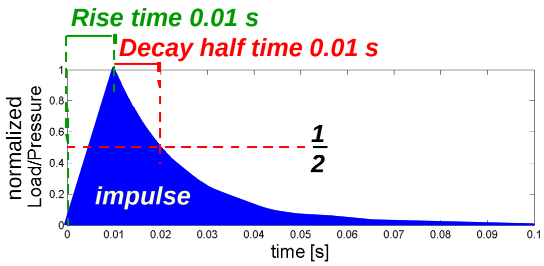


Figure 6-26: A typical example of a slam: a linear rise and an exponential decay

All measured impacts with a maximum larger than 300 kPa are plotted on top of each other in Figure 6-27. To facilitate comparison the slams are normalised with the maximum pressure and shifted in time such that the pressure maximum occurs at the same time.

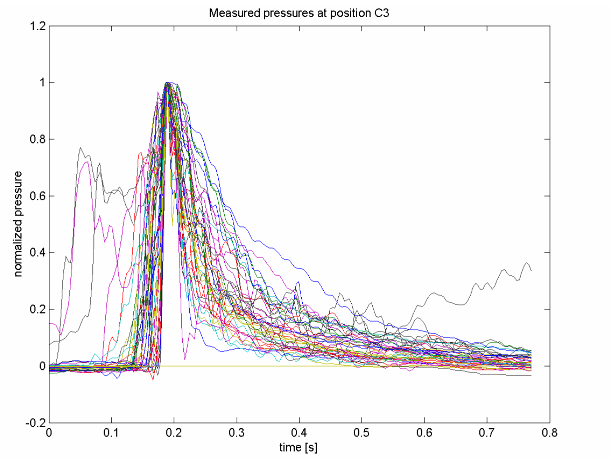


Figure 6-27: All measured impacts with a maximum larger than 300 kPa plotted on top of each other

A straight line from zero to the maximum level and an exponential decay from the top value back to zero can describe most slams. Figure 6-28 shows the slams, which are an exception to this rule. For these slams a second slam occurred directly after the first ones. This is also clear in the video snap shots in Figure 6-29.

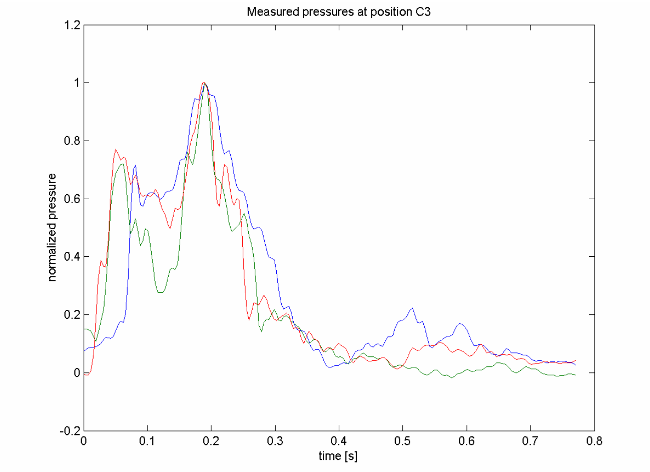


Figure 6-28: Time traces of special impact with a secondary impact

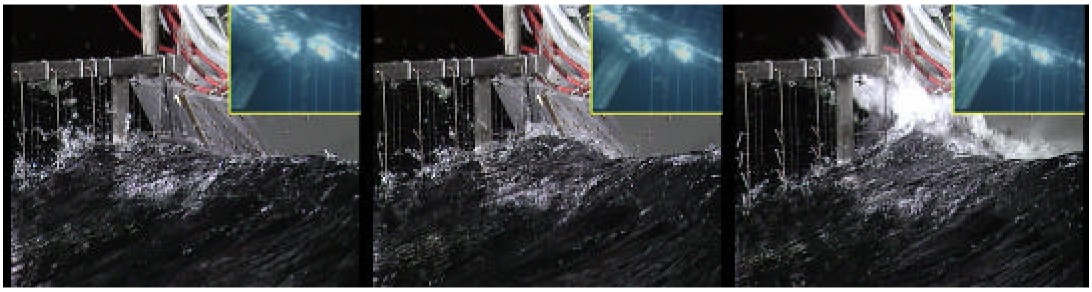


Figure 6-29: Video snapshots special impact with a secondary impact

As a linear rise and an exponential decay describe most slams, the rise time, maximum and duration together determine the slam. The variation in these values can be derived from the bold lines in the plot with the normalised pressures as shown in Figure 6-30.

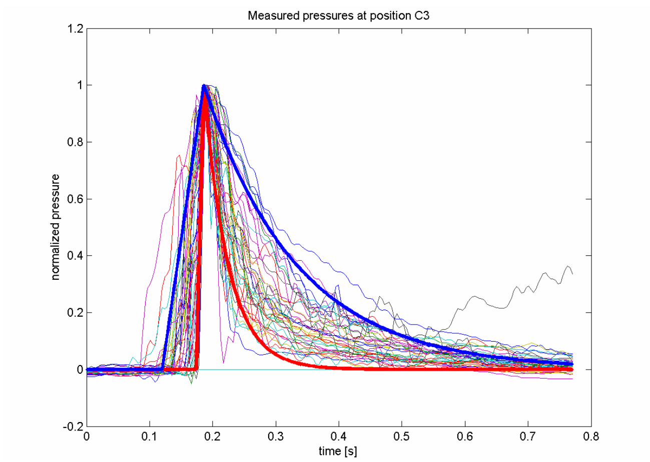


Figure 6-30: Idealised linear rise time ranges and exponential decays

For most of the slams the rise time is between 10 to 65 milliseconds can be expected. The duration of the slams defined at 10% of the maximum pressure value is between 0.1 and 0.45 seconds.

From the measured array of pressure on the schematic bow the pressure front or immersion velocity can be derived. This velocity is defined based on exceedance of half the maximum value for each transducer, as shown in the picture below. This choice of exceedance level makes the computer program capable of dealing with each slam and gives a better estimate then the exceedance of a threshold level. Sometimes this threshold level is exceeded due to the hydrostatic pressure only resulting in unrealistic small pressure front velocities. Figure 6-31 shows the derivation procedure.

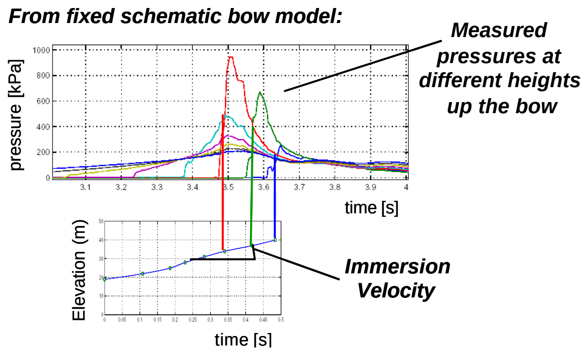


Figure 6-31: Derivation of pressure front or immersion velocity from the measured array of pressure transducers on the schematic bow

For some impacts the pressure front travels upward over the plate, while others occur more instantaneously. If we determine the time between the impacts over the height of the flat plate differences of 0.0 to 0.3 seconds can be found for two adjacent transducers. This corresponds to a velocity of 10 m/s and higher. For comparison the time derivative of the water level against the plate is determined, which has a maximum of approximately 16 m/s. Up to immersion velocities of 100 m/s the pressure time trace of the slam show the traditional shape (progressive slams), above this velocity only short sharp (instantaneous) slams occur.

Figure 6-32 shows the immersion velocities derived from a large number of slams on the flat bow in different environments against the corresponding values of d?/dt (measure of the wave steepness).

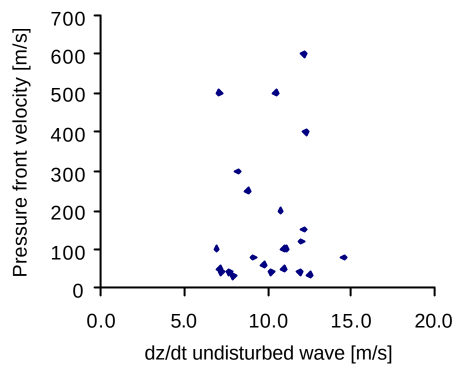


Figure 6-32: The immersion velocities derived from a large number of slams on the flat bow in different environments against the corresponding values of dζ/dt

As both progressing and instantaneous slams tend to occur at comparable values of dζ/dt, the probability of occurrence of the immersion velocities is used to calculate the chance of a simultaneous or progressive slam.

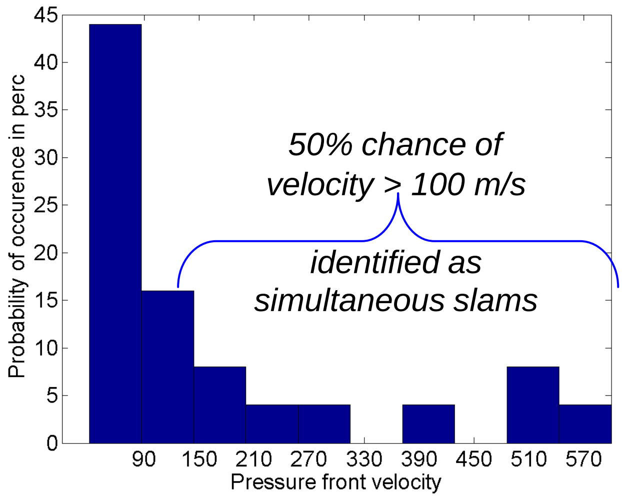


Figure 6-33: Difference in pressure front velocities between simultaneous and progressive slams

The spatial extent of the measured slams follows from spatial integration of the bow pressure downward from the maximum slam over increasing areas with steps of 1.5 metre, see Figure 6-34.

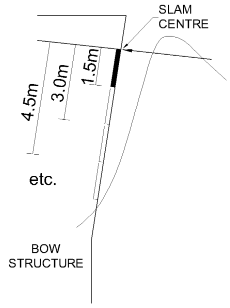


Figure 6-34: Spatial integration of the bow pressure downward from the maximum slam over increasing areas with steps of 1.5 metre

For the large impulse slams two typical situations were observed:

- The whole pressure time trace decreases with the integral over the area. This type of impulse is recognized as an instantaneous slam. The decrease of the pressure depends on the size of the pressure front and drops down fast during integration. When the pressure front is large (i.e. all transducer record the pressure peak), this can results in pressure oscillations due to air entrapment.

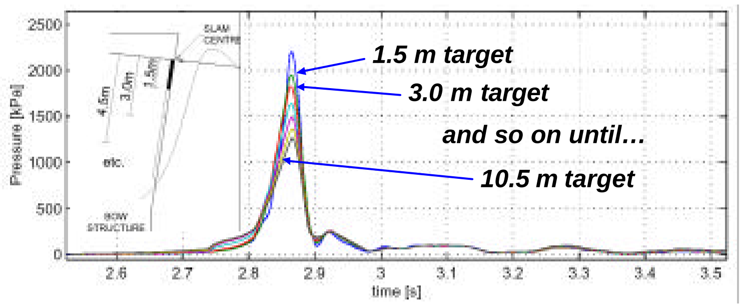


Figure 6-35: Instantaneous slam

- The rise time increases while the pressure impulse remains relatively constant. This second type of impulse progresses along the structure. Due to the pressure integration the rise time increases with the size of the area and the impulse remains approximately the same.

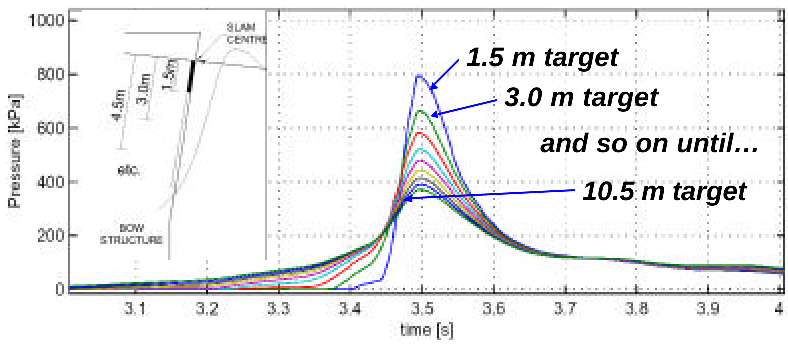


Figure 6-36: Progressive slam

These slams correspond to the sharp impact with short rise and decay times and the traditional impacts with a short rise time and more gradual decay, observed in the bow monitoring.

6.7 Dynamics response to impact loads

6.7.1 General

It is a characteristic of bow impact loads, particularly instantaneous slams, that pressure loads are applied very quickly. Dynamic response of the structure is therefore likely, requiring that dynamic amplification be included in any design calculations. The magnitude of this amplification depends on the rise time and decay time of the pressure, and the natural period of the target structure including any added mass of water moving with the plate panel.

Section 6.7.2 below recommends the extent of added mass acting with a structure, based both on theoretical calculations and observation. Section 6.7.3 gives a simplified approach to the calculation of dynamic amplification.

6.7.2 Added mass effects

Dynamic effects are potentially significant in determining the response of structural components to rapidly varying loads. To obtain an accurate estimate of structural natural period, both the stiffness and the mass of the component considered are required, since T = *2.p.Ö (M/K)*, where K is the component stiffness and M is its effective mass. Whereas K can be relatively accurately calculated for the structure, M is more difficult to assess as it is increased due to the presence of water in contact with the component (termed the added mass).

Added mass will tend to increase the natural periods of oscillation of the bow structure components and will modify their response to impulsive loads. The situation is complex, since bow immersion is typically progressive, causing added mass and resultant natural periods to vary throughout the slam duration. Separation (cavitation) can also occur and the water, particularly at the wave crest, may be aerated (of lower effective density).
The following figure has been derived theoretically, considering the deformed shape of panels of different aspect ratio and different levels of immersion. The thickness of the water layer that can be considered to move with the panel can be up to 80% of the stiffener spacing, but significantly less if the panel is at the water surface. For bow impact where the peak response is likely to be close to the water surface, lower values may be expected, perhaps in the region of 30-40% for horizontal stiffeners and plate, somewhat higher for vertical structures.

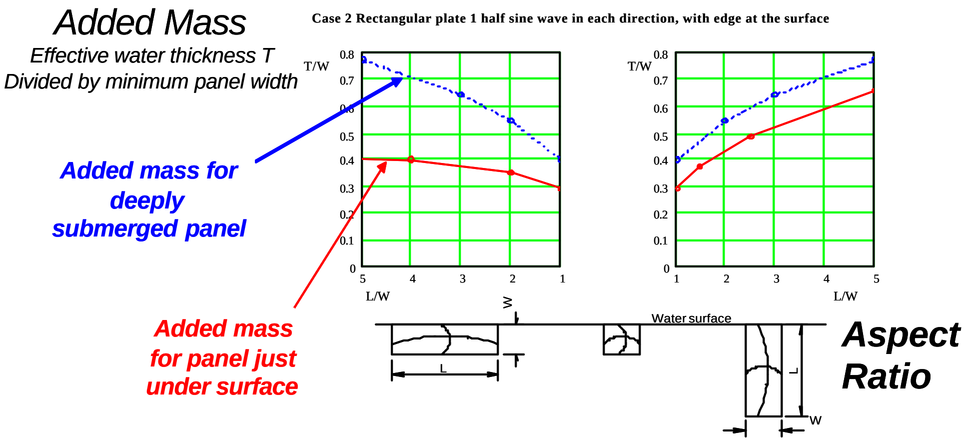


Figure 6-37: Estimates of added mass

Several sources of information are also available from full-scale and test data:

<u>BP Schiehallion Bow Monitoring System</u>

For vertical stiffeners on the bow of BP Schiehallion, the response to slap loading occurring on a stiffener was recorded by the monitoring system for several events. For slams on the upper gauges, the stiffener is largely immersed when the slam occurs. For impact on the lower gauges, it is only partially immersed. The natural frequency of vibration is around 22-28 Hz, which is approximately half the calculated frequency without immersion (50-60 Hz). This suggests that the effective mass (steel plus water) when immersed is approximately 4 times the mass when in air (steel only). The added mass of water is therefore approximately 3 times the steel mass.

Calculations of the mass per unit length of stiffener show that this ratio is achieved if the volume of water moving with the stiffener is approximately equal to $0.3hs^2$, where s is the spacing of the stiffeners and h is their height. In other words, the mean column of water moving with the stiffeners has an effective thickness (from the face of the plate) of about 30% of the stiffener spacing. This value is rather tentative, since: it has not been possible to directly correlate immersion depth with period; recorded periods vary somewhat; and it is uncertain exactly what length of the member is immersed.

MARIN Test Results

Natural periods in air and in water are available from the MARIN tests carried out within SAFE-FLOW for both the forecastle structure and the pressure panels embedded within the bow. Both show reductions in natural frequency between the in-air values and those deduced from the response data when in water. The following table summarises this data and includes a calculation of added mass as a percentage of the modelled mass of the component:

Component	**Natural Frequencies**		**Masses**		
	In Air	**In Water**	**Structure**	**% Increase**	**Added**
Upper Forecastle	4 Hz	3 Hz	2629 te	77%	2045 te
Lower Forecastle	16 Hz	8 Hz	2885 te	300%	8655 te
Upper Panel	70 Hz	56 Hz	6.3 te	56%	3.5 te
Lower Panel	65 Hz	50 Hz	6.3 te	69%	4.3 te

The thickness of the water layer moving with the panels is approximately 0.6 m, which is about 25% of the panel width (2.4 m). This is broadly consistent with the 30% value obtained from real time monitoring (see above).
The mass associated with the upper forecastle component is equal to about a quarter of the total displacement of the forecastle (taken as a semi-cylinder of diameter 45 m and height 11 m). This mass is lower than may be expected and is less than the corresponding value extracted from the Glasgow University tests (see below). The probable reason for this is because the forecastle is seldom fully immersed in the irregular waves that cause significant slam loading.

Mass associated with the lower forecastle component is significantly larger, greater than the displaced volume of the bow semi-cylinder (45 m diameter, 6.5 m high). This structure is clearly fully immersed and a higher mass would be expected, but the difference between the two results seems to be very pronounced.

Glasgow BP Schiehallion Tests

Natural periods are also available for the forecastle component of the BP Schiehallion model tested at Glasgow University [Xu and Barltrop (2004)]. As for the MARIN tests, these show a decrease in frequency of about 22% when immersed, but the scale mass of the forecastle component was greater (5500 te).

The added mass of water associated with this change in natural frequency is therefore approximately 3500 te, equal to 40% of the displacement of the forecastle. Since the structural mass of the full-scale forecastle is only 600 te, this would increase the natural period of its fore-aft vibration by a factor of about 2.5.

Summary

Full-scale and test results indicate generally similar results for small plates and these agree well with theoretical calculations for plates at or near the water surface. It is recommended that the mass of a thickness of water equal to 30-40% of the stiffener spacing should be added to steel mass for dynamic effects during wave impact. Significantly higher values are expected for fully immersed panels.

There is less certain agreement between added masses of fluid associated with the global forecastle, but if fully immersed by the slam, an added mass of 40% of the displacement of semi-cylindrical cross-section would seem to be appropriate.

6.7.3 Dynamic effects

The previous sections focussed on the bow impact loading. However, for the evaluation of ship-type offshore structures, the structural response under this impact loading determines whether a structure is able to survive a certain event.

The structural evaluation of bow slam loading and structural response is in principle a coupled hydro elastic problem. These coupled effects are likely to be most important when the natural period is longer than twice the duration of the slam. The full coupled problem has not been investigated in the present project. Instead a simplified approach has been used. This divides the slamming problems into two cases with respect to typical rise times and durations of bow slam loading:

1. The case where the natural period of the structural response, included added mass, is much shorter that the typical rise time of bow slam loading. In this case the bow slam peak load can be considered statically.
2. The case where the natural period of the structural response, included added mass, is approximately equal or longer than the typical rise time of bow slam loading, but shorter than the its duration. In this case the dynamic response of the structure will be important and a dynamic amplification factor or full dynamic analysis should be used to calculate the peak response

With the rise and decay time (and their characteristics) of the slam and the natural frequency of the structure the dynamic response can be calculated with a single degree of freedom mass spring system. Excluding hydro-elastic effects, the structural response can be described with the equation of motion:

$$(m+a)\cdot \ddot{x} + b\cdot \dot{x} + cx = F(t,x)$$

in which a constant damping (b), added mass (a) and mass (m) are assumed.

The ratio between the dynamic response and static response to the maximum pressure can be expressed as a Dynamic Amplification Factor (DAF).

Dynamic Amplification Factors for a linear pressure rise and exponential decay are given in Figure 6-38:

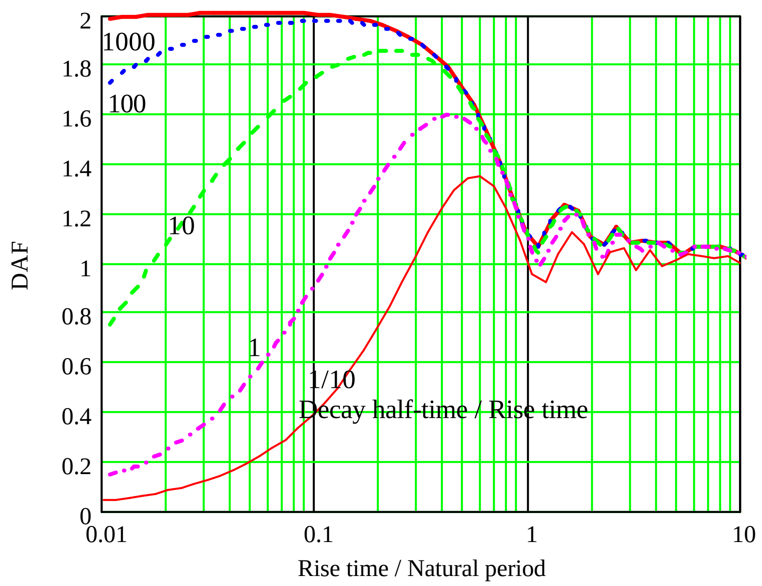


Figure 6-38: Dynamic amplification factors for a linear pressure rise and exponential decay

Based on this graph for each combination of rise time, half decay time and natural period the DAF can be calculated. The definition of the related parameters is shown in Figure 6-39:

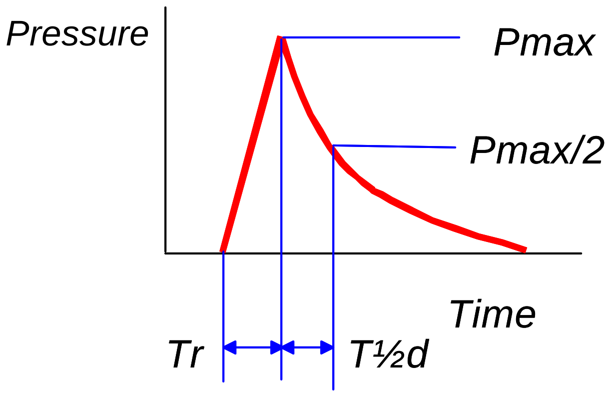


Figure 6-39: Definition of rise time (Tr) and half decay time (T½d)

7 WAVE IMPACT DESIGN GUIDANCE

7.1 Introduction

In Section 6, the nature of wave impact loading has been shown to be complex, with different types of impact pressure being identified that will result in different static and dynamic response effects on the various types of structure that comprise the vessel bow or hull (plating, stiffeners, frames, forecastle, etc.). A structural design methodology is required that predicts the probability of extreme loads and allows for likely response to these loads in deriving structural sizes.

Section 7.2 describes how the general principles proposed in Section 3 of this report apply to bow impact problems. Three methods of design are presented that allow bow impact to be addressed at different levels, from the detailed assessment of probability, through a load factor approach, to a simplified method that is suitable for conceptual design.

At the heart of the more detailed structural assessment methodologies is the BowLab program that has been developed during the SAFE-FLOW project for the calculation of impact pressure probabilities on different target structures. A summary of the program capabilities is presented in Section 7.3.

The BowLab approaches are considered too complex for preliminary structural design. A simplified approach using a ‘first estimation approach’ is introduced in Section 7.4.

Section 7.5 considers how calculated bow impact pressures may be used in design and analysis. Section 7.6 reviews specific problems in the design of different types of structure that are subject to bow impact load. Section 7.7 considers the potential for fatigue damage to structures and potential failure due to fracture.

7.2 Adaptation of general design methodology to wave impact

7.2.1 General

A general design methodology has been described in Chapter 3 of this report that allows design of FPSO structures for rare intense events, including bow impact. The methodology ties-in with typical code based approaches, in that:

- Limit States are clearly defined.
- ALS and ULS criteria may be shown to be satisfied.
- Different levels of design are considered, from concept through detailed to reassessment.
- Load factors are supportable based on reliability assessment.
- Equivalent static design pressures can be derived and directly compared with current rule based methods.

Serviceability, fatigue and fracture criteria should also be met, as discussed later. This sub-section will deal only with strength criteria.
The approach describes three levels of structural design, specifically:

- Method 1 – *A first principles reliability based approach* – This approach is most suited to the development of load factors for a particular vessel type in a particular environment, and to determine the probability of failure for existing structures that may not have been designed to current practice.
- Method 2 – *A limit state approach* – This method is most suitable for the detailed design of structures, where mean load factors are already known based on prior reliability assessment or from SAFE-FLOW.
- Method 3 – *A first estimation approach* – This approach is most suited to preliminary or conceptual design and uses simple empirically based design pressures to determine structural sizes.

The application of these approaches to the design of hull and bow structures against wave impact loading is discussed in the following sections.

7.2.2 Method 1 - A first principles reliability based approach

Figure 7-1 illustrates the use of a first principles reliability based approach, as applied to wave impact loading problems. The approach may be used directly to design structures to provide a certain maximum probability of failure, or more commonly to derive mean load factors for subsequent structural design.

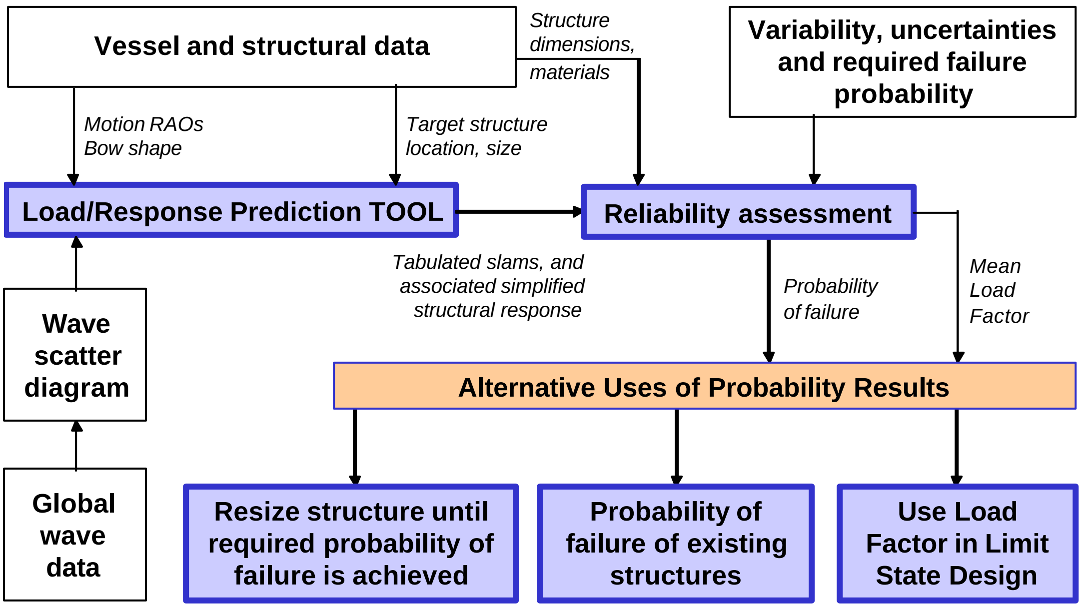


Figure 7-1: First Principles Reliability Design Approach for wave impact

The recommended procedure that should be adopted to design vessel structures subject to wave impact loading using this approach is as follows:

1. Obtain general vessel data, including operational draft, freeboards at locations of interest (bow, sides, stern) and motion RAOs. Where necessary, multiple drafts can be considered with corresponding freeboard and RAO data, separately considering the periods of time that the vessel spends in each condition, and summing the probability of damage to a specific structure. Operating restrictions can be accommodated by limiting sea states for particular draft scenarios.
2. Obtain sea state probability data (scatter diagrams) for the likely areas of operation of the vessel, extrapolated, if necessary, to provide suitable resolution in critical seas. If possible, remove sea states that will not contribute significantly to wave impact on the structure being considered.
3. Consider likely target structures (plate, stiffener, forecastle, etc.) at critical locations on the hull and determine their structural sizes, materials and natural periods.
4. Enter the vessel, structural target and scatter diagram data into the BowLab load probability prediction tool, developed and provided by the SAFE-FLOW project.
5. Perform a first level reliability assessment based on equivalent pressure probability data, target structure pressure data, and expected structural and material variability. A MathCad spreadsheet has been derived for this purpose. This data can be used in several ways:
   - Based on a required failure probability (derived with due regard to the consequence of this failure and to rule requirements), derive the expected mean load factor that will give this required failure probability, for use in subsequent structural design in accordance with Section 7.2.3, below.
   - For reassessment of existing vessels, derive the probability of failure that will result for a given structural design and check this against acceptable levels.
   - Resize the structure iteratively, repeating the reliability calculation, until the required probability of failure is achieved and adopt this as the required structural design.

It is recommended that the first of the above alternatives be used in general design, because the BowLab program contains only a simplified structural simulation. Whilst this is adequate for the development of load factors (essentially a comparative problem), it is of limited application in design.

7.2.3 Method 2 – A limit state approach

Figure 7-2 illustrates the use of a Limit State Approach for wave impact loading problems. Where the vessel and the environment are covered by the SAFE-FLOW project, the approach may be used directly to design a target structure on the hull of an FPSO using mean load factors given herein. Alternatively, mean load factors should be derived from previous calibration by First Principles Reliability Analysis.

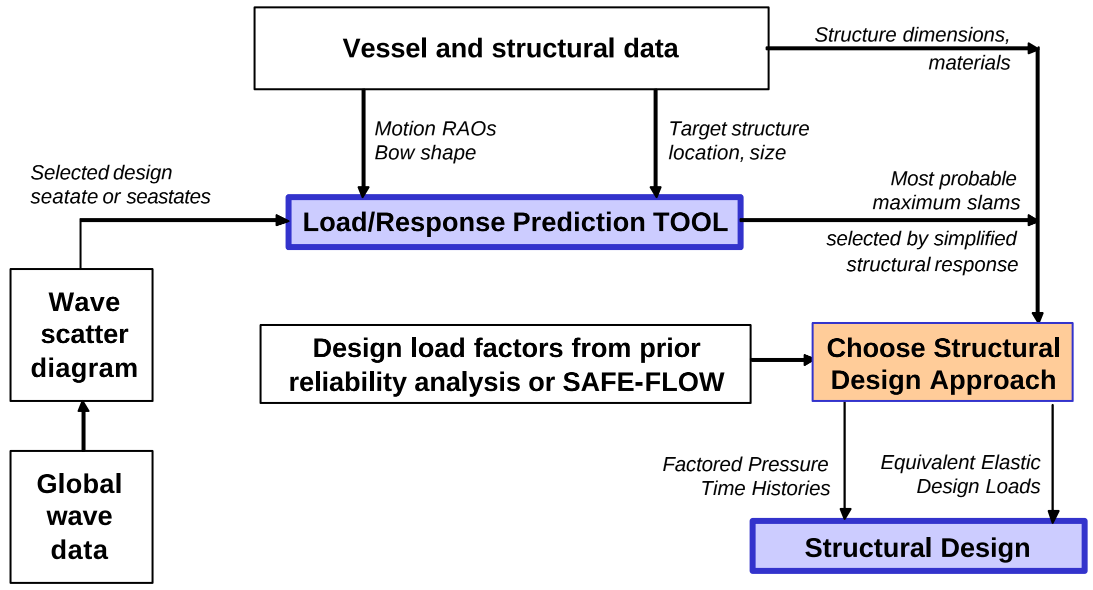


Figure 7-2: Limit state approach for wave impact

The recommended procedure that should be adopted to design vessel structures subject to wave impact loading using the Limit State Approach is as follows:

1. Obtain general vessel data, including operational draft, freeboards at locations of interest (bow, sides, stern) and motion RAOs. A range of likely drafts would normally be considered, with associated RAOs and sea state probability (e.g., the vessel may rarely operate at extreme draft, for which less severe seas may be considered).
2. Obtain sea state probability data (scatter diagrams) for the likely areas of operation of the vessel, extrapolated, if necessary, to provide suitable resolution in critical seas. Determine critical sea states consistent with the required probability of occurrence (typically 10-4 for ALS conditions and 10-2 for ULS conditions) with due regard to peak wave and vessel pitch periods. Where in doubt, consider several sea states with different periods but having the same probability of occurrence as the highest 10-4 0r 10-2 sea state, to search for the worst loading.
3. Consider likely target structures (plate, stiffener, forecastle, etc.) at critical locations on the hull and obtain their preliminary structural sizes, materials and natural periods.
4. Enter the vessel, structural target and selected sea state data into the BowLab load probability prediction tool, developed and provided by the SAFE-FLOW project.
5. Apply the Mean Load Factor (from SAFE-FLOW or calculated from previous First Principles Reliability Analysis) to the pressure data from BowLab and perform structural design or strength assessment using one of two methods:
   - Using transient Finite Element or equivalent Analysis (FEA), with hydrodynamic added mass appropriate to the response, determine the response to factored time histories of typical bow impact loading.
   - Determine the Equivalent Elastic Design Load (EEDL) for the structure from the factored BowLab pressures and the type of structure being considered. Use this EEDL to design the structure elastically.

7.2.4 Method 3 – A first estimation approach

Figure 7-3 illustrates the use of a first estimation approach for the design of hull structure under wave impact loads. The approach involves the direct calculation of design pressures for use in a simple elastic design method, similar to that provided in the majority of rule based design approaches.

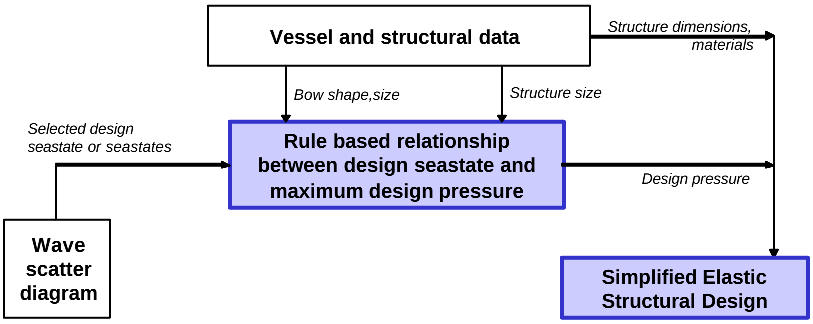


Figure 7-3: First estimation approach for wave impact

The methodology is very simple:

1. Critical design sea states are selected that are representative of the extreme seas to which the vessel will be subjected. Several such sea states may be selected if there is doubt as to which is critical.
2. Design coefficients should be derived based on the bow shape and size, and on the size of the structural component being designed (panel, stiffener, forecastle, etc.).
3. The equivalent design pressure should be derived based on the above factors, wave celerity in the selected sea state and an empirical relationship between pressure and input data.
4. Structural design is performed based on the above equivalent design pressure suitably factored by the likely dynamic amplification of response.

7.3 Summary of step by step approach in BowLab

7.3.1 General

It is clear that bow impact probabilities, magnitudes, elevations and the response of target structures are complex functions of the following:

- The maximum vertical velocities found within individual waves for critical sea states.
- The motion of the vessel at these times, giving the elevation of the maximum impact up the hull structure.
- The location of the target structure relative to this impact.
- The empirical relationship between vertical velocity and impact impulse magnitude, and the scatter in this relationship.
- The size of the target structure, dictating how rapidly the impact passes over the structure.
- The natural period of the target structure, and the resultant dynamic response to each impact load.

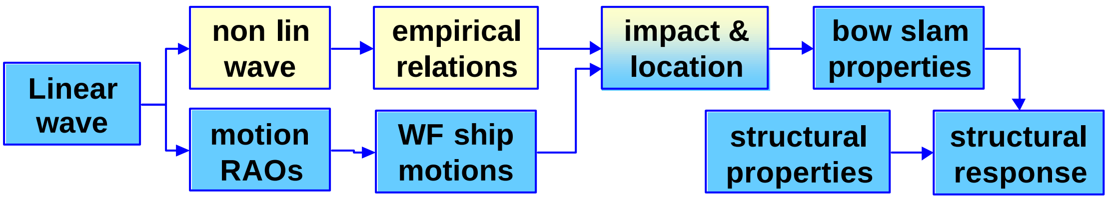
Because of this complexity, a load-response prediction tool has been developed that determines the probability with which structural responses occur within selected sea states. This tool has been created in Matlab and is named ‘BowLab’.

BowLab operates in the time domain, generating individual waves using the second order wave theory. If the magnitude of the peak vertical velocity within each wave is great enough, then a slam ‘event’ will be recorded and structural response at the target structure will be evaluated. A record will be kept of the equivalent uniform pressure on the target structure that would give the same response as the actual impact event. This allows the probability of equivalent static pressure to be accumulated and displayed.

7.3.2 Summary of step by step approach in BowLab

The background of the load-response prediction method in BowLab was presented in Chapter 6. Below a description will be given of the step by step approach in the program. Where needed, some additional detail will be provided.

To predict the probability with which structural responses occur within one selected sea state, BowLab uses the following steps:



1. A linear wave spectrum is chosen, based on significant wave height, spectral period and spectral shape

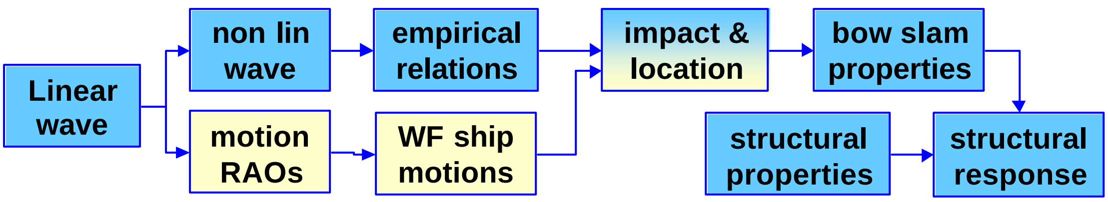
2. The second order wave theory is used to predict a time trace of the second order wave elevation and vertical free surface velocity (d?/dt)

3. Based on the empirical relation between the vertical free surface velocity (d?/dt) and the Impulse the distribution of impulses in this sea state is determined. The impulse magnitude (I) and its probability (P) is expressed:

   $$I = c_I \cdot \frac{d\zeta}{dt} \text{ in which } c_I = 7.6\frac{\text{tonne}}{\text{m}^2}$$

   $$P = 0.19633 \cdot \left(\left\langle \frac{d\zeta}{dt} - 3.631 \right\rangle^1 - \left\langle \frac{d\zeta}{dt} - 8.724 \right\rangle^1 \right)$$

   in which $\langle x\text{-}a \rangle^1$ is a ramp function equal to 0 for a<x and equal to x-a for a>x.



4. Based on the motion RAOs (pitch and heave) the linear motions of the ship are determined for the linear wave underlying the second order wave according to the following procedure:

    - For a time trace of the linear wave the motions of the vessel are calculated.
    - With this linear wave non-linear wave components are calculated and added to the time traces according to second order theory.
    - Combining the time trace of the non-linear wave and the linear ship motions gives a relative wave motion in front of the vessel.

5. For all the impulses determined in this sea state, the combination of the linear motions and the related crest height (according to second order theory), the slam centre can be determined.

    Figure 7-4 visualizes the procedure. The yellow ship is moving on the blue linear seas. While the red line indicates the second order wave which determines the impact size and location.

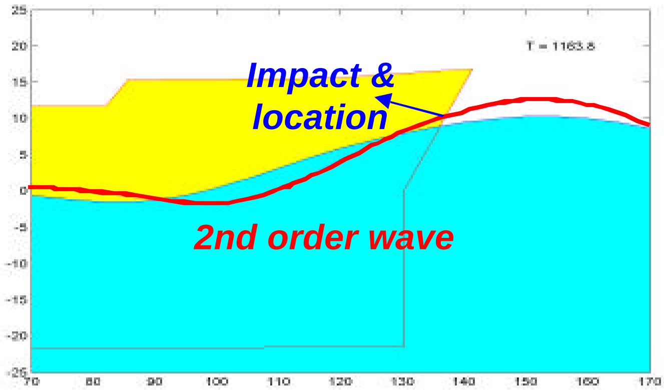
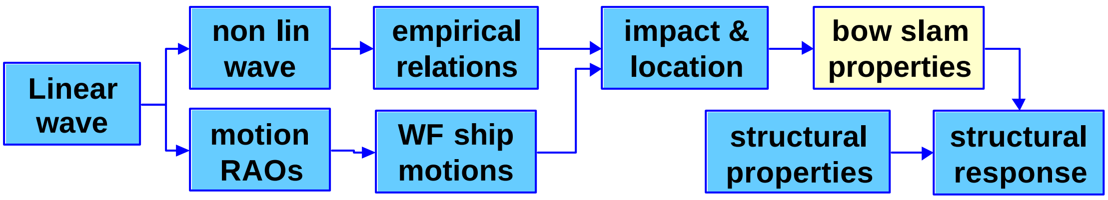


Figure 7-4: The position of the wave impact on the bow is determined by the linear motion response combined with the second order crest height



So far the method focussed on the local impact pressures on the bow and on its position. However, to design the plating and stiffeners in the bow area this is not always sufficient to determine an accurate response with resulting stresses. The monitoring on the Schiehallion bow allows for this comparison between pressures and stresses. Based on these results the pressure duration, immersion velocity and spatial extend are considered critical. These parameters were derived from the model tests with the schematic bow.

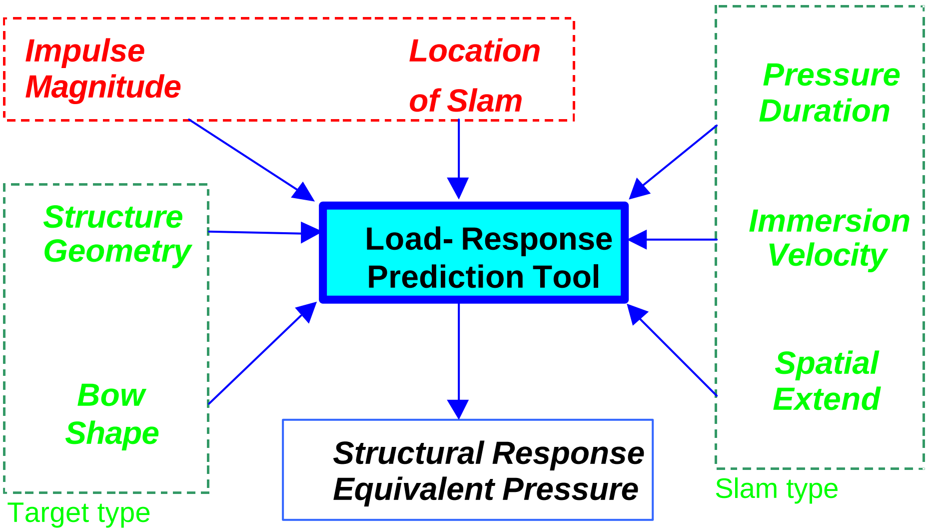


Figure 7-5: Bow impact characteristics

6. Different **immersion velocities** tend to occur at comparable values of vertical free surface velocity. Therefore the probability distribution of the immersion velocities is determined. This distribution is independent of the slam size. The average distribution is shown in Figure 7-6. This figure shows two types of slams:
   - The blue bars on the left result in slams that progress over the bow. The duration of the resulting load time traces on a bow segment are direct related to these velocities.
   - For velocities above 100 m/s (the red bar) an instantaneous slam occurs. The duration of the resulting load time trace is independent off the target size and thus independent of the immersion velocity.

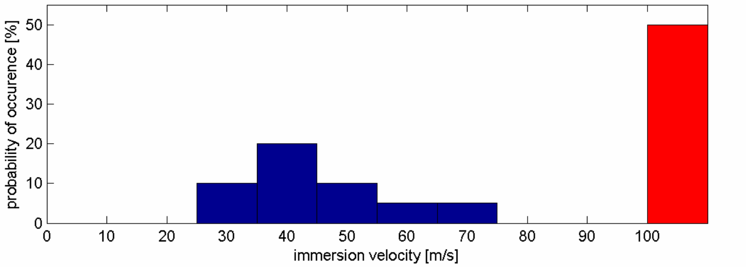


Figure 7-6: The probability distribution of the immersion velocities

Each identified wave slam is split in six different cases with the following probabilities:

- 50% probability on an instantaneous slam
- 10% probability on a progressive slam with immersion velocity of 30 m/s
- 20% probability on a progressive slam with immersion velocity of 40 m/s
- 10% probability on a progressive slam with immersion velocity of 50 m/s
- 5% probability on a progressive slam with immersion velocity of 60 m/s
- 5% probability on a progressive slam with immersion velocity of 70 m/s

7. For the schematic bow with the large number of pressure transducers the spatial extent is derived by integrating the bow pressures downward from the maximum slam over increasing areas, with steps of 1.5 metre. For the instantaneous and progressive slams this resulted in the following curves:
   - Instantaneous: The whole pressure time trace decreases with the integral over the area. This type of impulse is recognized as an instantaneous slam. The decrease of the pressure depends on the size of the pressure front and drops down fast during integration. The example below shows the results for an local impulse of 113 kPa.s.

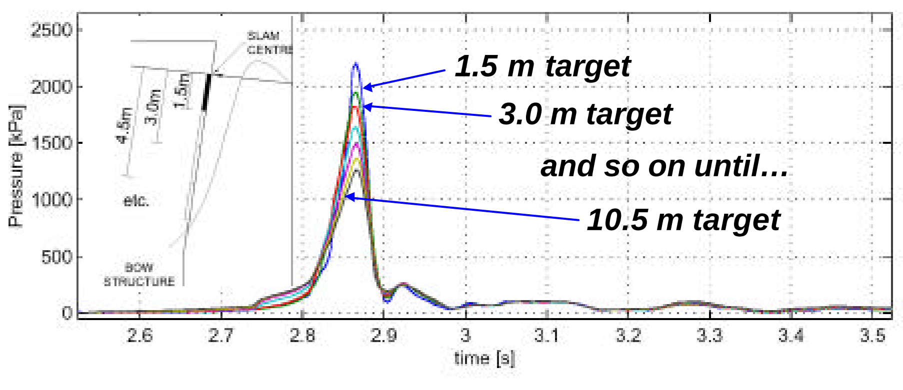


Figure 7-7: Used time trace of instantaneous slam

   - Progressive: The rise time increases while the pressure impulse remains relatively constant. This second type of impulse progresses along the structure. Due to the pressure integration the rise time increases with the size of the area and the impulse remains approximately the same. The example below shows the results for an immersion velocity of 40 m/s and local impulse of 130 kPa.s.

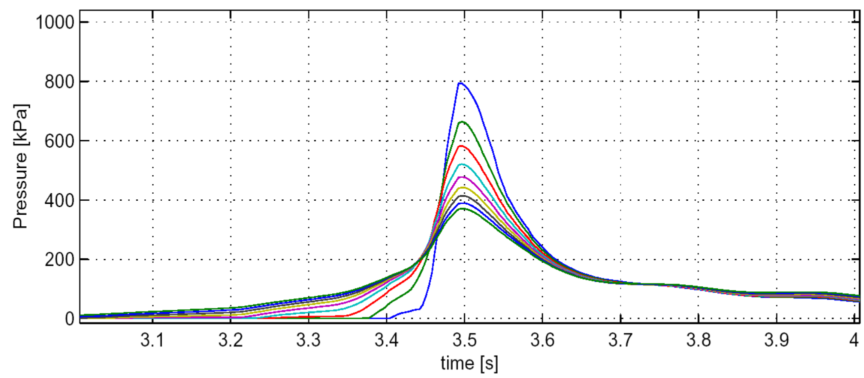
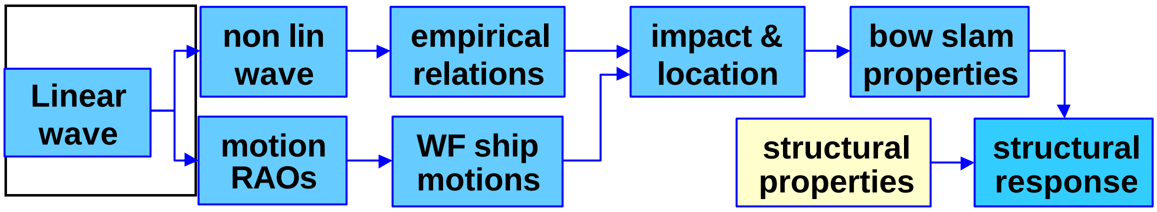


Figure 7-8: Used time trace of progressive slam

For each calculated impact the above curves are scaled to represent the correct impulse and immersion velocity as follows:

$$Pp(t,h) = Pp_m\left(\frac{v}{40}\cdot t, h\right)\cdot\frac{I}{130}\cdot\frac{v}{40} \quad \text{and} \quad Pi(t,h) = Pi_m(t,h)\cdot\frac{I}{113}$$

In which $Pp_m(t, h)$ and $Pi_m(t, h)$ are respectively the measured progressive and instantaneous slam time traces for target height h. I is the calculated impulse and v the immersion velocity. Note that five different values of v exist for each slam with different probabilities.



8. With the curves from step 7 and the location of the slam centre the equivalent static pressure on a specified panel can be determined as follows:

$$P_i(t) = \max\left(\frac{Pi(t,a+h)\cdot(a+h) - Pi(t,a)\cdot a}{h}\right)$$

in which a is the distance between the upper side of the panel and the slam centre and h is the height of the panel. This equation is only valid if the slam occurs above the panel. When the slam occurs below the panel the resulting pressure on the panel is zero. When the slam centre is located on the panel at height (b) above the lower side of the panel the following equations hold:

$$P_i = \max\left(\frac{Pi(t,b)\cdot b}{h}\right)$$

Equivalent equations hold for the calculation of the equivalent static pressure from an instantaneous slam. This way each wave slam results in six values for the equivalent pressure with a summed probability of occurrence equal to probability of the wave.

9. The width and curvature of the panel can result in a reduction of the equivalent static pressure. For very small panel width the results can be considered two dimensional and multiplication of the pressure with the width of the panel gives the design load. For larger panels the curvature of the bow becomes important. Even for a flat plate the average pressure reduces with the width of the panel (w_{panel}) due to local wave effects. The average pressure on a flat plate can be expressed as:

$$p_{flatplate} = p_{local}\cdot\left(1-\frac{w_{panel}}{w_{max}}\right)$$

in which p_{local} is the local pressure calculated with the method described in the previous section and w_{max} is the maximum width of the panel and equals 100 metre.
For curved panels the radius of the curvature R accounts for a further reduction of the pressures. An empirical formula is fitted through the measurements performed in Glasgow and documented in Xu and Barltrop (2004):

$$p_{curvedplate} = p_{local}\cdot\left(\frac{0.37}{2.6\cdot\sqrt{a}}\right) \qquad \text{for } w_{panel} < 0.8\text{ R}$$

$$p_{curvedplate} = p_{local} \cdot \left(\frac{0.37 \cdot \sqrt{0.412}}{2.6 \cdot a} \right) \qquad \text{for } w_{panel} > 0.8\ R$$

in which $a = \frac{w_{panel}}{2 \cdot R} + 0.012$

The above equations show a tendency that the curved plate formula becomes infinite as the panel width becomes small relative to the curvature. Since one would expect the curved bow factor to be always less than the flat plate value, the pressure for a curved plate is limited to the flat plate value. The resulting factor between the local pressure and the pressure average over the width of the plate is shown in Figure 7-9.

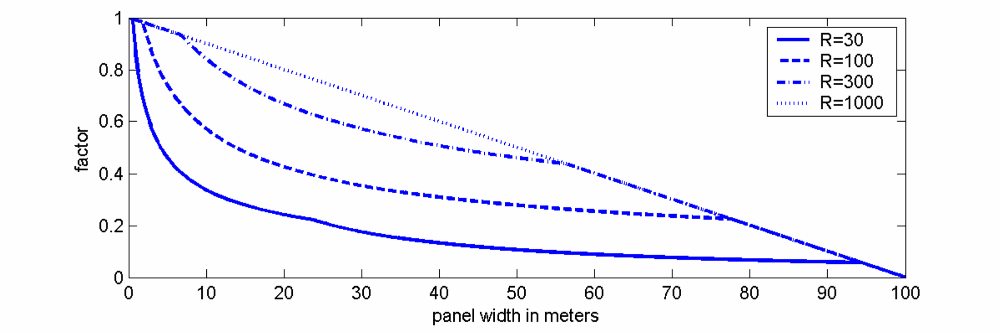
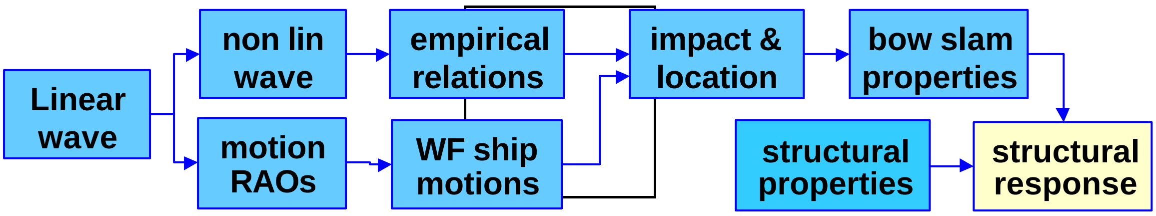


Figure 7-9: Pressure reduction factors for curved panels



10. The equivalent dynamic pressure follows from the static pressure and the structural response. The ratio between these is expressed as a Dynamic Amplification Factor (DAF) which depends on the :

- Rise Time
- Decay Time
- Natural Frequency

The rise and decay time results are taken from the time traces in step 8, while the natural frequency is an input to the program. Some guidance on the calculation of natural frequencies and added mass is given in Chapter 6.

For a typical large local slam the rise and decay times are about 0.1 sec and for a panel natural period of about 1/30 sec the DAF will be 1.05. It is worth noting that dynamic amplification factors can be less that 1.0, as in Figure 7-10, if the structure has a long period and does not have time to respond to the impact load before it starts to reduce:

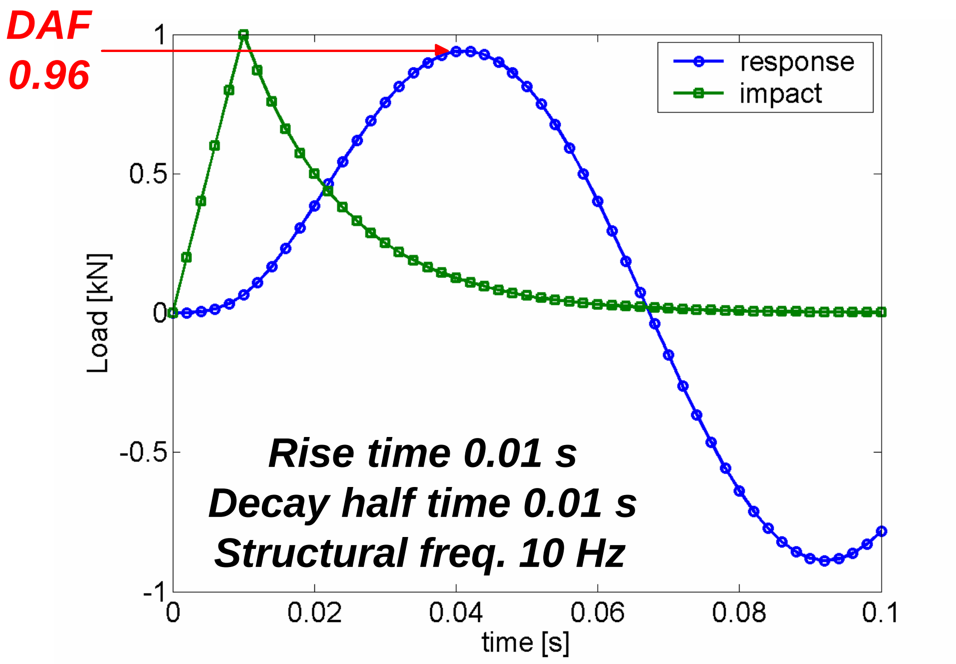


Figure 7-10: Example of a DAF smaller than 1

Another example is provided in Figure 7-11 for a stiffened panel with an immersed natural frequency of 20 Hz and bow slam rise and decay times of 0.01 and 0.03 seconds. The rise time divided by the natural period is 0.2, reading the graph vertical to the inclined line with the correct ratio between the decay half time and the rise time of 3, results in a dynamic amplification factor of 1.66.

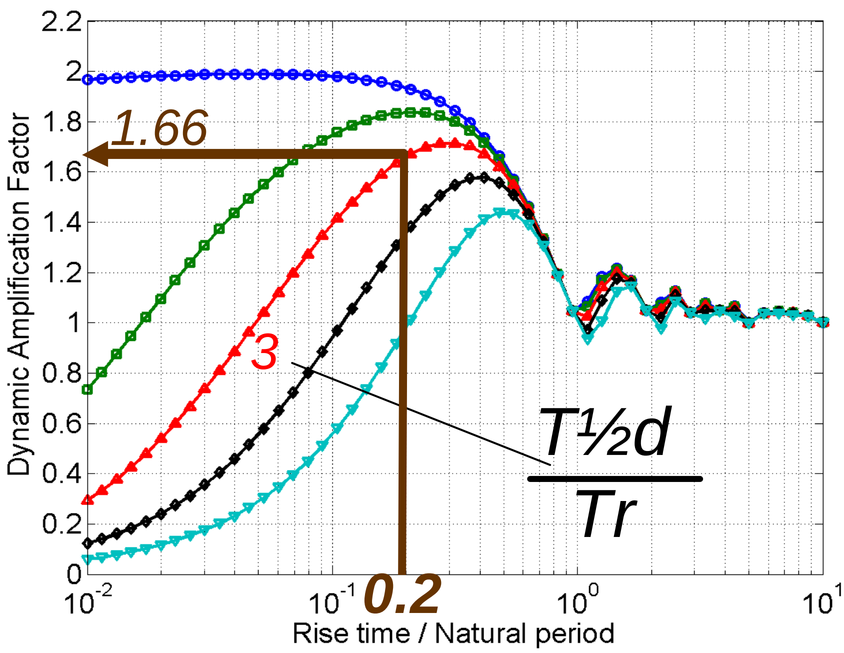


Figure 7-11: Example of a DAF of 1.66

7.3.3 BowLab examples and program input/output

The following figure shows an example of the input to and the output from the BowLab tool:

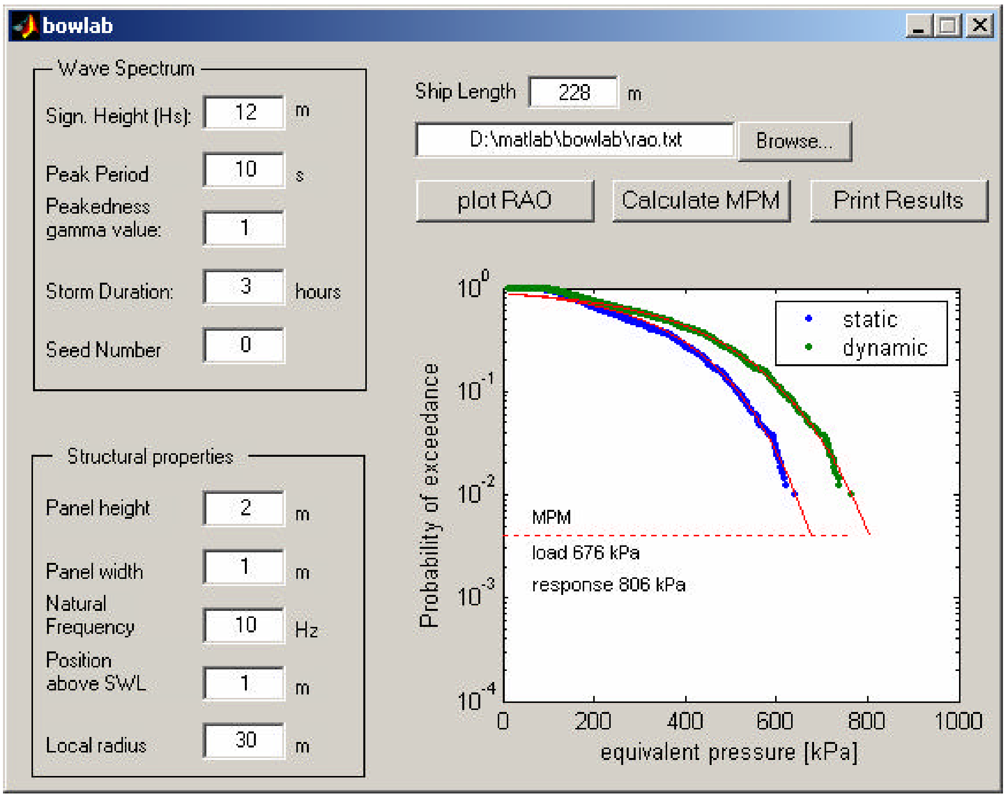


Figure 7-12: Graphical User Interface of BowLab

In this particular example, an H_s = 12 m, T_p = 10 s sea state has been processed for a peakedness of 1 and a duration of 3 hours. The seed number is an integer which sets the state of the random generator for the phases of the waves. A 228 m long vessel with specified motion RAO data (in the rao.txt) is being considered. These RAOs can be taken from the databases in the GreenLab program or determined with diffraction analysis

Equivalent pressure loads have been developed on a 2 m high, 1 m wide structural panel within a 30 m radius bow structure. The centre of this structure is located 1 m above MSL (Mean Surface Level) and its natural frequency is 10 Hz.
Results indicate that a most probable maximum (MPM) equivalent pressure up to 806 kPa (81 m head of sea water) can occur once in a 3 hour sea state, when structural dynamics are included and based on a fit to the probability data.

Some examples of results of BowLab are presented below.

Figure 7-13 shows the effect of the wave steepness on the slam loads. Up to a significant wave height of 5 metres no slams occur on the specified panel. Above this value the loads increase with the wave height as expected.

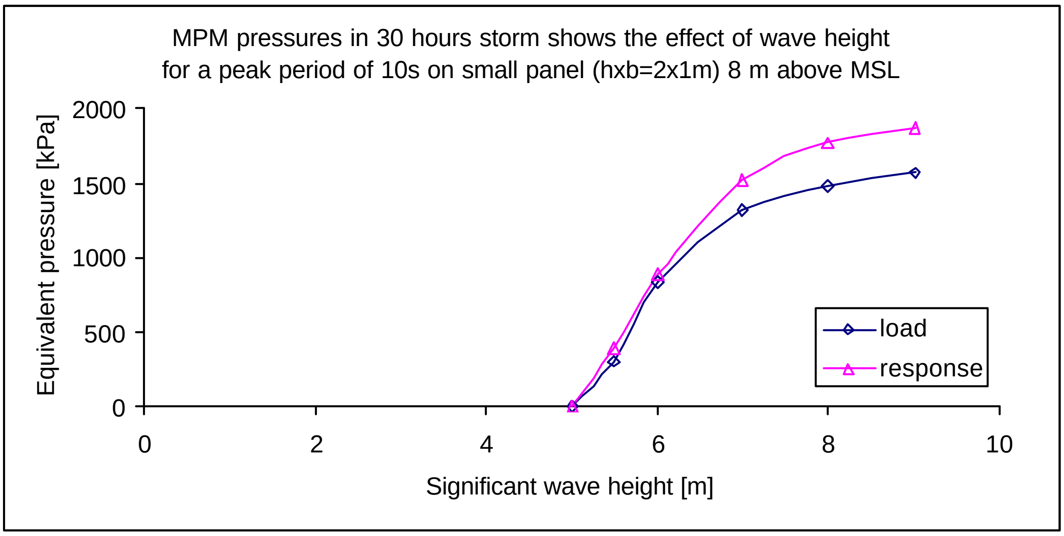


Figure 7-13: Increase of equivalent pressures with wave height

The tool can also be used to check the sensitivity of the loads on the panel position. Figure 7-14 shows the results for a 2 m high, 1 m wide structural panel within a 30 m radius bow structure. In these calculation, an H_s = 12 m, T_p = 10 s sea state has been processed for a peakedness of 1 and a duration of 3 hours. The centre of this structure is shifted from 1 to 21 m above MSL with steps of 2 m.
The results indicate that a most probable maximum (MPM) equivalent pressure up to 1600 kPa (160 m head of sea water) can occur once in a 3 hour sea state, when the panel centre is close to the significant wave height above the MSL (close to the crest heights in the wave).

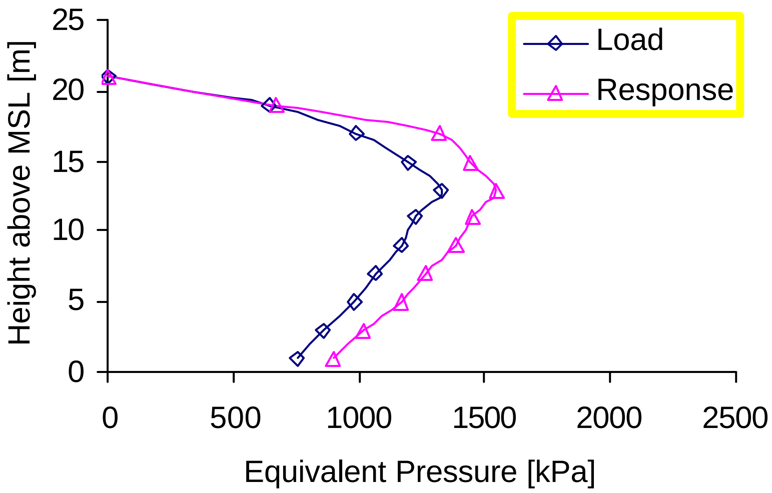


Figure 7-14: Effect of panel position above Mean Surface Level (H_s = 12 m)

7.4 The use of BowLab in reliability calculations

The BowLab program can be used to estimate a suitable long-term distribution of slam response pressures on any relevant structure and this in turn can be used as input to a reliability assessment of the design. The WP4 Technical Reports describe this process including comments on variations that might be expected with selection of values within the range of appropriate input parameters. The essential steps are summarised as follows:

- Prepare a suitable scatter diagram representative of the vessel location and long-term period of interest. The method developed on SAFE-FLOW uses a 10,000 year scatter diagram as the basis for the reliability assessment.
- Input the height of the lowest point of the design detail above mean water level together with a set of pitch and heave RAOs to represent the vessel's response at the appropriate draft. Where the vessel takes on a number of drafts during normal operation and its response varies significantly with draft, it may be necessary to implement this calculation for several drafts and assemble a distribution of results according to the proportion of time the vessel is liable to be in each draft.
- Input the size, location, natural frequency and bow curvature at the detail of interest.
- Define each scatter diagram entry and hence each BowLab simulation according to significant wave height, peak period and spectral peakedness, gamma.
- Select a run duration that gives (statistically) stable output values for sea states in which slams are predicted. It may be necessary to increase run duration if a short run predicts only a small number of slams. Likewise, it may be necessary to change the random number seed if some difficulties with internal convergence are advised.
- Make a series of runs for all sea states for which a number of slams are predicted.
- Take the output of BowLab for each short-term run, notably, the number of slams, the duration of the run, the three Weibull parameters for the short-term distribution of equivalent slam response pressure and the most probable maximum as input to a long-term response model. At this point in the calculation a review of results in the form of most probable maxima in 100 and 10,000 years, an appreciation of numbers of slams, levels equivalent static pressure contribution that may be generated by each sea state and the percentage probability that each sea state may give rise to the maximum value may be a useful check. The Mathcad software needed to implement this stage and the display of some typical results is detailed in the WP4 Technical Reports. Two sample tables are included below:

Probability that each sea state will cause the most probable maximum slam

Hs m \ Tp -> s	2	4	6	8	10	12	14	16	18	20	22	24	26
24													
22													
20													
18								0.0	0.0				
16							0.0	0.0	0.0	0.0			
14						0.0	0.1	0.2	0.0	0.0			
12					0.0	0.2	1.9	1.1	0.1	0.0			
10					0.2	5.5	10.0	2.1	0.1	0.0			
8				Supressed	5.7	26.8	14.6	1.6	0.0	0.0	0.0		
6			0.0	1.0	8.5	17.2	3.0	0.0	0.0	0.0	0.0	0	
4		0	0	0	0.0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0

Note: An entry 0 indicates, on the basis of the available scatter diagram, that at least one sea state occurs with the given Hs and Tp but no slams are predicted.
An entry of 0.0 indicates that there is 0 or a small probability (<0.1%) that a slam will occur in this sea state.
A blank square indicates no sea state is expected with these parameters within the 10000 year scatter diagram.
Analysis data for sea state with Hs 8m, Tp 8s, supressed - sea comments in text
Value of most probable maximum response (pressure) in 10000 years is 2.36*10^3 kilo Pascals (approximately 23 bar)
Value of most probable maximum response (pressure) in 100 years is 1.93*10^3 kilo Pascals (approximately 19 bar)

Probability that 10,000 year, most probable maximum slam will occur for each sea state.

Most probable maximum slam response pressures for each scatter diagram entry taken singly
Units kPascals

	Tp -> s	2	4	6	8	10	12	14	16	18	20	22	24	26
Hs m														
24														
22														
20														
18									1248	1195				
16								1386	1388	1333	1045			
14							1517	1537	1430	1355	966			
12						1617	1714	1566	1639	1638	1147			
10						2221	1911	1804	1829	966	637			
8					Supressed	2332	1796	1331	691	359	131	0.0		
6				1506	685	546	651	190	0.0	0.0	0.0	0.0	0	
4			0	0	0	0.0	0	0	0	0	0	0	0	0
2		0	0	0	0	0	0	0	0	0	0	0	0	0

Note: An entry 0 indicates, on the basis of the available scatter diagram, that at least one sea state occurs with the given Hs and Tp but no slams are predicted.
An entry of 0.0 indicates that there is 0 or a small probability (<0.1%) that a slam will occur in this sea state.
A blank square indicates no sea state is expected with these parameters within the 10000 year scatter diagram.

Value of most probable maximum response (pressure) in 10000 years is 2.36*10^3 kilo Pascals (approximately 23 bar)
Value of most probable maximum response (pressure) in 100 years is 1.93*10^3 kilo Pascals (approximately 19 bar)

Most probable maximum slam response pressure for each scatter diagram entry, taken on its own.

- Form the long-term response by summing the short-term response distributions weighted by the proportion of the total number of slams to which the structural detail is predicted to be susceptible for each short-term BowLab run.
- Fit a two parameter Weibull distribution to the top one third of all slams so predicted and use this as the long-term distribution of equivalent, static response pressures as input to the reliability model.
- Using an appropriate structural model of the design detail of interest, evaluate the reserve strength ratio as the ratio between the equivalent static response pressure that generates an unacceptable plastic failure of the structure and the equivalent pressure that causes first yield or an unacceptable quasi-elastic failure of the detail. This ratio is used following, not during the initial sizing calculation - next step.
- Provide an initial size of the design (determine appropriate thicknesses and dimensions) such that the elastic failure corresponds to the most probable maximum response pressure in 100 years. Where a distribution of design pressures is available (due, for example, to several different slam force time series), the method developed in other parts of this project would recommend that the initial size is based on a mean pressure as determined by said distribution.
- Introduce a load factor in the initial sizing equation such that it the factor is greater than unity, the sizes are increased.
- Using all available information about distributions of load, uncertainty in pressure, modelling uncertainty, structural dimensions and yield parameters calculate the annual probability of failure of the detail. Determine the value of the load factor that enables the structure to achieve the required probability of failure during the 10,000 year event (or equivalent where some other return period has been selected according to perceived criticality of the risk or available level of redundancy).
- Apply the safety factor to the initial sizes.

It is important to make a realistic estimate of the modelling uncertainty in the above calculation. Where this is not available, the modelling uncertainty must be increased to account for this underlying variability in results.

When this methodology is applied to shear design of a bow stiffener, as detailed in the WP4 Technical Report, the following table was generated as an example of a sensitivity calculation.

Parameter varied	Range of variation	Range of safety factor required to satisfy design requirement
Ship design life (years)	**20** - 30	**0.994** - 0.985
Material yield strength (MPa)	**350** - 450	**0.994** - 0.994
Standard deviation of Yield strength (kPa)	**5**% - 6%	**0.994** - 1.000
Coefficient of variation of modelling uncertainty	**0.23** - 0.34	**0.994** - 1.282
Required annual probability of failure	10^{-3} - $\mathbf{10^{-4}}$	0.821 - **0.994**
Reserve strength ratio	1.1 - **1.3**	1.178 - **0.994**

Sensitivity study for reliability of bow stiffener against failure in shear

Similar trends are evident in this analysis as were observed in the previous cases. The basic parameters used in this case lead to a safety factor that is less than unity. This may be explained, partially, by the higher reserve strength ratio that is adopted here, but note the lower probability of failure (10^{-4}) that has also been required.

As with the green water calculations, the methodology is seen to be robust for variations in material parameters and to exhibit the expected trends with changes in modelling uncertainty and reserve strength ratio.

Finally, given the fact that explicit variability in slam pressures has not been modelled in this case, the result for the higher coefficient of modelling uncertainty would seem more appropriate for general design, in line with the comments given above.

It is worth noting that the above pressures are high. Designing elastically for greater than 100 m head and plastically for 200 m head is onerous. However, this is generally consistent with the results of the Schiehallion Bow Monitoring (see WP3 Technical Report) and therefore does not appear to be a significant overestimate of the design requirement.

Significantly reduced loads may be expected on a bow structure that is less bluff than Schiehallion. Even so, pressure loads are still expected to be severe. Much of this is thought to be due to significant scatter in the experimental data. Even single waves can produce significantly different pressures on adjacent rows of pressure sensors. Local water surface slope, which is chaotic and random, probably accounts for a lot of this local variation.

7.5 First estimation approaches

7.5.1 First estimation approach based on SAFE-FLOW methodology

In the BowLab program the second order wave and the resulting linear ship motion are calculated in the time domain. This can be time consuming. To come with a faster first estimate, WP1 and WP2 derived an alternative approach for the extreme slam load based on the following simplifications:

- The maximum impacts occur in the crest of the waves. Therefore it is assumed that the maximum load on a specific structure occurs in the sea state with significant height equal to the elevation of the structure.
- A 3-parameter Weibull distribution was derived to predict the distribution of the vertical free surface velocity, see the WP1 Technical Report for details.
- The maximum equivalent static pressure is largest for an instantaneous slam. As a conservative approach the MPM load is calculated with the equations for a static equivalent pressure of an instantaneous slam.

In WP1, Weibull, Rayleigh and Gumbel distributions were fitted to empirical distributions of maximum vertical free surface velocities of the waves both recorded in the sea and simulated. It was confirmed that the cumulative distribution of the maxima of the derivative of water surface elevation fits well a 3-parameter Weibull distribution. Figure 7-16 gives an example for the Draupner full-scale data.

The Weibull probability of exceedance is defined as:

$$P_{exc}(X) = e^{-\left(\frac{X-\theta}{\alpha}\right)^{\beta}}$$

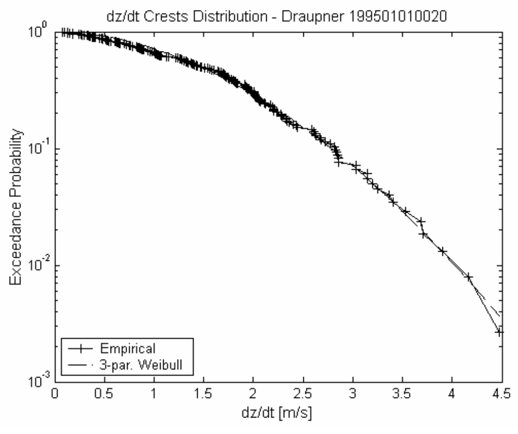


Figure 7-16: Distribution of maximum vertical free surface velocities of waves

The least square method was used to estimate the parameters that best fitted to some basin and field data and also to extensive numerical simulations of sea states. The estimates are given in the table on the next page for different values of the shape factor γ in the JONSWAP spectrum formulation. It is observed that for all data the Weibull shape parameter grows with significant wave height.

H_s [m]	T_p [s]	$\gamma = 1.0$			$\gamma = 2.0$			$\gamma = 3.3$		
		α	β	θ [m/s]	α	β	θ [m/s]	α	β	θ [m/s]
6	7	2.38	1.56	0.00	2.22	1.50	0.00	2.23	1.60	0.00
6	10	1.78	1.64	0.13	3.23	2.62	1.94	1.76	1.78	0.28
6	12	1.92	2.03	0.57	2.48	2.44	1.35	1.54	1.89	0.25
8	8	2.82	1.52	0.00	2.70	1.54	0.00	2.59	1.54	0.00
8	10	2.92	1.80	0.70	2.21	1.52	0.00	2.92	1.96	0.91
8	12	2.12	1.72	0.22	1.93	1.64	0.11	2.50	2.07	0.85
8	14				2.48	2.22	1.11			
10	9	3.19	1.55	0.00	3.43	1.66	0.48	2.89	1.47	0.08
10	11	2.60	1.53	0.00	2.41	1.46	0.00			
10	13	2.29	1.61	0.26	2.06	1.56	0.06	2.47	1.83	0.67
10	15	1.84	1.60	0.00	3.08	2.29	1.66	2.62	2.23	1.10
12	10	3.97	1.69	0.59	3.34	1.54	0.00	3.22	1.56	0.00
12	12	5.59	2.30	3.54	2.76	1.60	0.00	3.69	1.96	1.29
12	13	2.83	1.62	0.24	3.42	1.83	1.32	2.32	1.50	0.00
12	14	3.02	1.84	0.78						
12	17	3.41	2.25	1.72						
14	10	3.69	1.44	0.00	3.60	1.51	0.00	3.51	1.52	0.00
14	13	3.92	1.75	0.96	3.24	1.61	0.31	3.69	1.77	1.10
14	14	4.19	1.98	1.61	3.58	1.81	1.10			

With the table above the most probable maximum for the equivalent static pressure can be calculated with the following steps:

1. Determine the steepest sea state occurring with wave height equal to the height of the upper side of the panel from the waterline. This is the smallest peak period with entry larger than zero in the scatter diagram.
2. Based on the Weibull parameters for this sea state, the (most probable) maximum vertical free surface velocity (d?/dt) can be determined.
3. This directly gives the related impulse according to:

$$I = c_I \cdot \frac{d\zeta}{dt} \text{ in which } c_I = 7.6\frac{\text{ton}}{\text{m}^2}$$

4. Parallel to the time domain approach (step 7 and 8) the maximum equivalent static pressure for an instantaneous slam can be written (see the WP2 Technical Report for details):

$$p_{local} = p_m \cdot \frac{I}{113\text{kPa.s}}$$

in which p_m is a function of the height of the panel as shown in Figure 7-17.

Combined with step 3 the pressure can be written with the following equation:

$$p_{local} = \frac{p_m}{b_I} \cdot \frac{d\zeta}{dt} \text{ in which } b_I = 14.9\frac{\text{m}}{\text{s}}$$

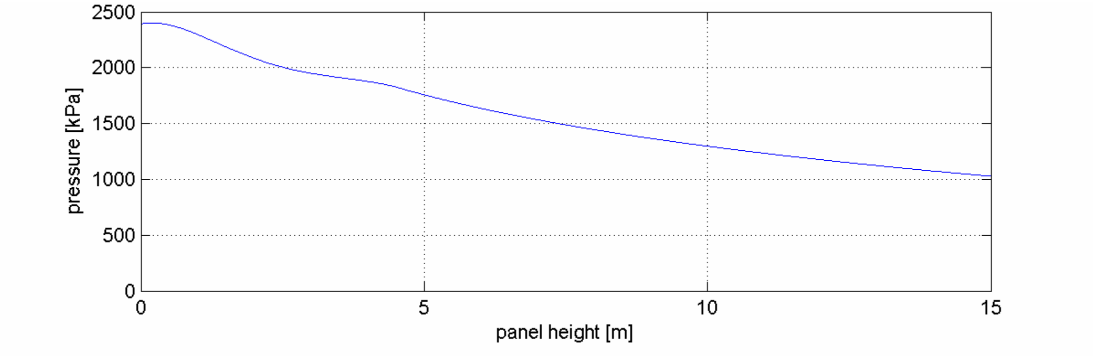


Figure 7-17: Maximum equivalent static pressures as function of panel height

5. The width and curvature (R) of the panel result in a reduction of this pressure equal to step 9 in the time domain approach. The factor is shown in Figure 7-18.

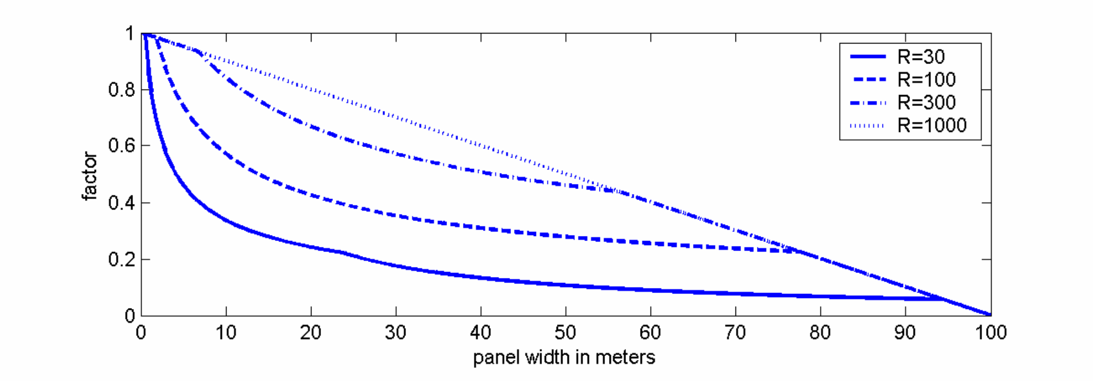


Figure 7-18: Pressure reduction factors for curved panels

6. Finally the equivalent dynamic pressure follows from the static pressure and the structural response. The ratio between these pressure is expressed as a Dynamic Amplification Factor which depends on the rise and decay time and the natural frequency of the panel. The instantaneous slams indicate that the rise and decay times are equal. This reduces the design diagram in step 10 of the time domain approach to the curve shown in Figure 7-19.

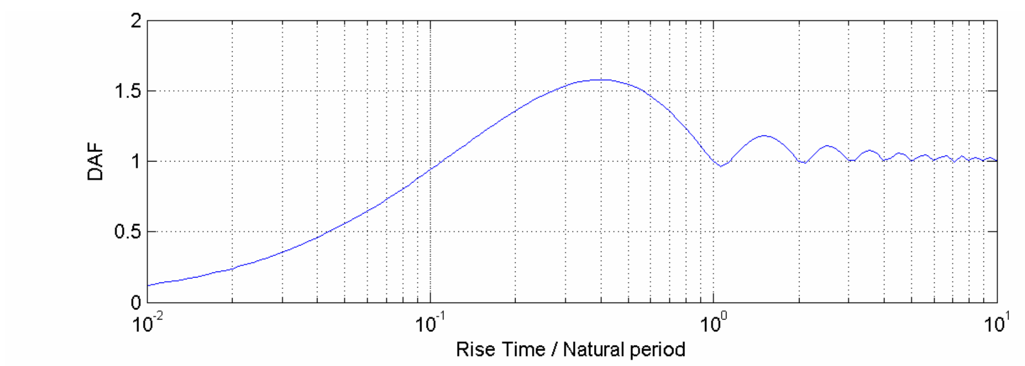


Figure 7-19: Dynamic amplification factors for equal rise and decay time

This approach requires a mean load factor to be applied.

7.5.2 Alternative first estimation approach

An alternative approach for was derived by the Department of Naval Architecture & Marine Engineering of Glasgow & Strathclyde Universities, NAME, (previously Naval Architecture & Ocean Engineering of Glasgow University), see Barltrop and Xu (2004). Based on their research, the following design equation for the most probable maximum quasi-static pressure head (m) in a 3 hour spread -sea storm of given H_s and T_z was estimated as:

$$h = \frac{1}{g} CE(S)F(W)G(Z)V_S^2 \; DAF$$

where:

h design pressure (m sea water).

C coefficient determined from tests to best fit the results. The mean C value (with dynamics and calculated DAF) was found to be 0.39.

E(S) coefficient that depends on the sea-state steepness:

$$S = Hs\left(\frac{2\pi}{g{T_z}^2}\right)$$

$$E(S) = \begin{cases} 1 & \text{if } S \geq 1/12.5 \\ (12.5S)^{3.5} & \text{if } S < 1/12.5 \end{cases}$$

F(W) coefficient that depends on the panel projected width (normal to the wave) (W), and panel horizontal curvature. The horizontal curvature may be found by fitting a circle of diameter D which should be a tangent to the plating at the extreme loaded width W:

$$F(W) = \begin{Bmatrix} \left(\frac{W}{D} + 0.012\right)^{-0.5} & \text{if } \frac{W}{D} \leq 0.4 \\ 0.642\left(\frac{W}{D} + 0.012\right)^{-1} & \text{if } \frac{W}{D} > 0.4 \end{Bmatrix}$$

Alternatively, and especially for head seas, it may be more convenient to fit an ellipse. If an ellipse is fitted to the plan bow shape 2A is the longitudinal ellipse diameter, B is the transverse ellipse diameter and $e = 2A/B$. As for the circle, the ellipse should be fitted to be a tangent to the plating at the extreme loaded width W:

$$F(W) = \begin{Bmatrix} \left(\frac{We}{B} + 0.012\right)^{-0.5} & \text{if } \frac{We}{B} \leq 0.4 \\ 0.642\left(\frac{We}{B} + 0.012\right)^{-1} & \text{if } \frac{We}{B} > 0.4 \end{Bmatrix}$$

The definitions of A, B, D and W are shown in Figure 7-20.

The formulae imply that pressure reduces with the square root of the loaded width for smaller loaded widths and reduces inversely with width for the larger loaded widths.

In reality 3-D bow shape effects will be important but are not included in these formulae, other than through the calibration against real bow shapes.

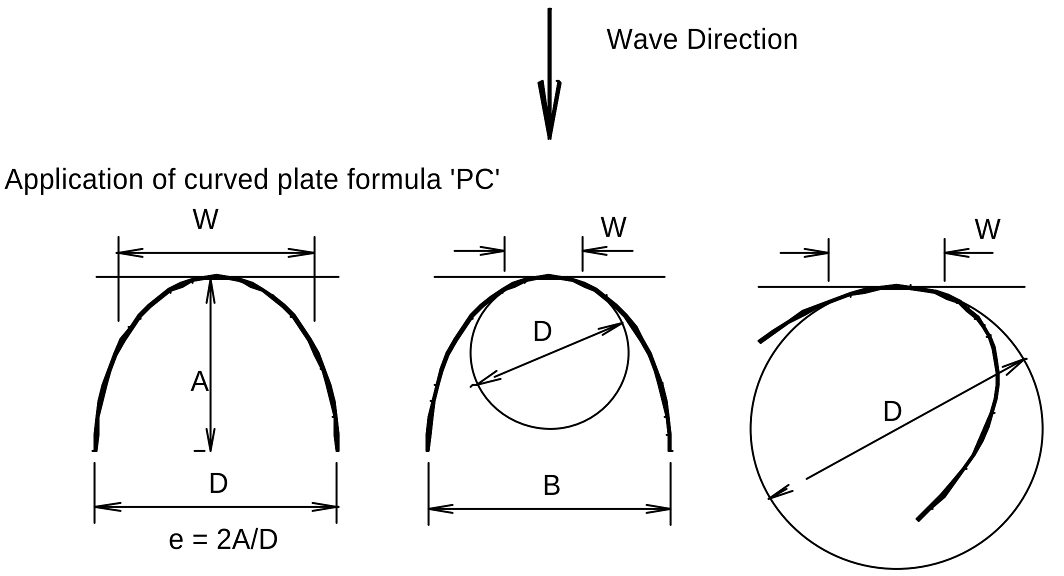


Figure 7-20: Definitions of A, B, D and W

G(Z) coefficient that depends on the panel height (Z) and the zero-crossing wavelength of the sea:

$$G(Z) = \left(\frac{Z}{Lz/12} + 0.4 \right)^{-0.5}$$

(Note H_s may be substituted for *Lz/12*. This gives a slightly better fit to the data and requires a smaller value of *C*, 0.355 instead of 0.39).

V_s notional slam velocity.

$$V_S = \sqrt{c_z c_z \tan(\theta) * 1.07} = 1.37 c_z$$

c_z is the zero crossing celerity $(gT_z/2\pi)$
? is the typical maximum angle between the wave front slope and the horizontal
? is fixed at 60 degrees in this formula. The effect of lesser slopes is taken into account by *E(S)*.
1.07 allows for predicted spread-sea celerity exceeding c_z.

The measured against predicted pressures for various different panel or bow segment sizes are shown (with dynamics excluded, i.e. *DAF* = 1) in Figure 7-21.

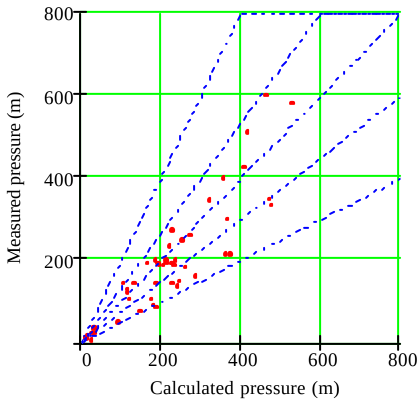


Figure 7-21: Measured and calculated pressures, excluding dynamics

To calculate the DAF it is necessary to know the immersed panel natural frequency and to estimate rise and decay times for the pressures.

The following empirical formulae are proposed for the slam rise time and time for the pressure to decay to half the peak value:

$$t_{rise} = 0.062e\frac{D}{c_z}\left(\frac{Z}{Hs}\right)^{0.3}\left[1+\left(\frac{W}{D}\right)^4\right]$$

$$t_{halfdecay} = 0.022e\frac{D}{c_z}\left(\frac{Z}{Hs}\right)^{0.3}\left[1+10\left(\frac{W}{D}\right)^4\right]$$

The dynamic amplification factor is then calculated assuming a linear rise time and exponential decay.
The measured against predicted 'quasi static pressures' (with dynamics included) for various different panel or bow segment sizes are shown in Figure 7-22.

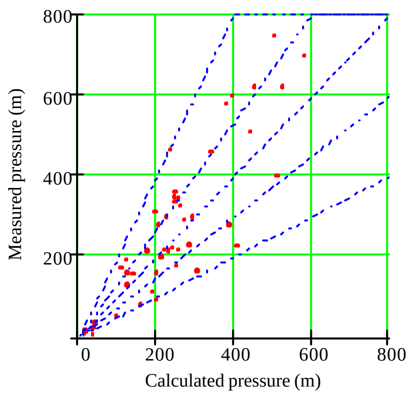


Figure 7-22: Measured and calculated pressures, including response dynamic amplification

The coefficient of variation of the measured/calculated dynamic response was 0.38. A large part of the uncertainty is associated with the dynamic response. The coefficient of variation of measured/calculated applied pressure is much lower at 0.27, implying that the coefficient of variation on the dynamic calculation is $\sqrt{0.38^2 - 0.27^2} = 0.27$ so, for the experiments, the uncertainty in the prediction of the dynamic amplification of the response is about the same as the uncertainty in the evaluation of the pressure.

Note that a mean load factor is required with this approach.

7.6 Detailed structural design analysis

7.6.1 General

The derivation of mean load factors to apply to obtain design pressures can be achieved with the BowLab program coupled with reliability analysis, using the Method 1 approach. Once these mean load factors are defined for appropriate conditions and structural target types, detailed structural design can be performed in accordance with Method 2 using suitably factored pressure loads.
Examples of structural design aspects are provided in the WP3 Technical Report on wave impact loading. Significant aspects of structural design against wave impact loading are highlighted in the following text.

7.6.2 Alternative design approaches

For the selected sea state, the BowLab program produces an Equivalent Design Pressure over the specified target structure that is representative of the maximum simultaneous total loading to which it is subjected. This pressure includes the effects of dynamic response for the first mode only. It does not consider any non-linear response that may result from local yielding, tripping or buckling at extreme load.
This is justifiable in the derivation of load factors, since the same simplifying assumptions are being made throughout the reliability calculation and the load factor is effectively derived by dividing one simplified response by another. For structural design, however, more detailed analysis may well be required where non-linear response is likely at extreme load.
As discussed in Chapter 3, either ULS or ALS conditions may govern structural design for wave impact. Where the extreme loads occurring during rare intense Accidental Limit State conditions are significantly more severe, the structure may be designed inelastically to withstand the load. Since inelastic design is complex and time consuming, a simpler approach using equivalent elastic design loads is also considered.
Three different approaches to structural design are therefore described in this section:

- If the designer is confident that the structure being considered can be assessed accurately enough by simple equivalent pressure loading, then simple beam calculations will suffice.
- Where more detailed analysis of more complex structures is required, then the structure may be analysed for the expected critical temporal and spatial variation of load, typically using Finite Element Analysis methods.

- Where ALS conditions are being checked for which the structure need not remain elastic during the impact event, then the designer need not analyse the structure using complex material and geometric non-linear analysis. In such cases, he/she may instead design the structure elastically using an equivalent elastic design load.

7.6.3 Equivalent load design

If the equivalent design pressure from BowLab is used directly (with the application of a suitable factors), then the structure may simply be analysed and designed as if it were subjected to a uniform static pressure of the given magnitude:

- Dynamic effects do not need to be considered as these are already allowed for by input of a natural period into BowLab.
- Spatial and temporal effects do not need to be considered as these are incorporated into BowLab by the definition of the target structure size.

If ALS conditions (10^{-4} annual events) govern, then no load or material safety factors are required but the appropriate Mean Load Factor should be applied. Inelastic design requires the structure to remain essentially intact after the event with no rupture, although plastic response, limited buckling and yielding is permitted. It is recommended that strains in the structure should not be permitted to exceed 5%, to reduce the possibility of fracture.

7.6.4 Transient load design

Detailed assessment of a structure may be undertaken using representative temporal and spatial variations of load. This approach is recommended where the structure is not a simple stiffener or deck plate structure, or where there is uncertainty about its dynamic behaviour, or its ability to carry loading in excess of yield.
Numerical details of typical impacts are presented in the WP3 Technical Reports. Use can be made of the two typical slam events presented earlier in this Chapter: the progressive slam event that passes over the target structure and the instantaneous slam event.
This data may be used to reproduce electronically a ‘design’ slam event. However, these records must be scaled to represent the magnitude of the design impact event expected on the target structure. This can be achieved as follows:

- From the BowLab output, for the sea state giving the worst slam, the most probable maximum slam impulse (area under pressure time history during slam) can be obtained.
- The magnitude of the two slam events can be scaled to suit this required impulse, adjusting pressure magnitudes to suit.
- An appropriate Mean Load Factor (and for ULS, a load safety factor) should be applied to this loading to achieve a suitable probability of failure.
- The resultant pressure time histories may be applied to the FEA model or other simulation and resultant response obtained.

The two slams are chosen to be representative of typical events, but investigation of other slam durations may be undertaken based on this data by stretching or contracting the time scale for the two events, to represent different immersion velocities up the bow. If this is done, the pressure magnitudes should also be altered to maintain the correct impulse magnitude.

An example of this methodology for the FEA model in Figure 7-23 is given in the WP3 Technical Reports.

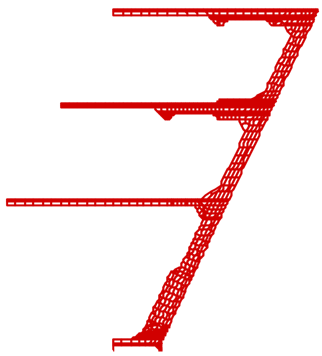


Figure 7-23: FEA Model of bow used for investigation of immersion velocities

7.6.5 Equivalent elastic design

If the ALS condition governs, then the structure may be designed inelastically for the applied load. As an alternative to this complex analysis procedure, it is possible to design the structure for a reduced level of load termed the Equivalent Elastic Design Load (EEDL). This level of load is calculated by dividing the design load by an Ultimate Strength Ratio (USR).

Equivalent Elastic Design Load = (Equivalent Inelastic Design Load) / (USR)

USRs are set so that elastic design of the structure at the EEDL results in the same structural sizes as inelastic design at the original full design load.

There is no reason why transient load design may not be treated in the same way, dividing the temporal and spatial distributions of load in Section 7.6.4 by the USR to produce load time histories for linear finite element analysis. Stresses should be checked against yield, but in such cases, it may be necessary to accept local yielding where this is caused wholly by discontinuities in the structural model.

USRs vary with structural type and design. Typical values are presented in the Technical Reports and in Section 7.7, by structure type. These values have been based on analysis of different typical structural configurations. Where specific structural components are not covered by SAFE-FLOW, then the designer can derive specific values using similar methods.

In such analysis, suitable strain limits should be imposed on the structure to prevent fracture at extreme load. The structure should also be compact, else local buckling and tripping of flanges should be considered in the analysis. Specific considerations for different types of structure are presented in Section 7.7.
Highest USRs are produced for deck plate, since first yield of this structure type occurs in bending, yet the major load carrying capacity of the structure is produced by membrane action.

The USR of stiffener and frame structures depends very significantly on the extent of end fixity. Encastré structures are expected. Failure of such structures will not occur until plastic hinge formation at mid-span and at the ends of the member, hence relatively large USRs are obtained. If the stiffeners were not continuous at their ends, then significantly lower values would be obtained.
Shear at the ends of stiffeners should be treated separately from bending, since there is considerably less reserve strength associated with this type of loading. Failure of one end of a member in shear is normally catastrophic, whereas failure in bending by formation of plastic hinges is relatively ductile. Hence significantly lower USRs apply to shear failure.

USRs for supporting structures (decks, internal walls, etc.) depend on how stocky these are and their susceptibility to buckling.

The reserve strength of global structures such as forecastles very much depends on their geometry, relative member sizes and the redundancy of load paths back into the main hull structure. Recommendations are given in Section 7.7.6.
It should be noted that it is always conservative to use a USR of 1.0. This is equivalent to designing the structure elastically for the Accidental Limit State.

7.7 Design of specific structural types

7.7.1 General

Different structural types likely to be subjected to wave impact loads are discussed in this section. Typical sizes, natural frequencies and Ultimate Strength Ratios are given, to aid with design. Consideration is also given to the effect of structure size on impact load magnitude.

Wave impact loads will predominate on the bow structure, but are possible in other regions, such as the sides, fore-foot and stern. Standard naval architectural details are common in each location. Stiffeners tend to frame between transverse frames or bulkheads and are usually continuous through cut-outs in the transverse frames, with welded brackets to allow easy fit-up and provide a shear strength. Frame flanges are often discontinuous (sniped back) at intersections, with brackets being introduced to maintain continuity.
For the connections between major items, full or partial penetration butt welds are uncommon, fillet welds being used almost exclusively. Welds are typically sized using a rule- based approach, with throat sizes set to a certain proportion of the plate size, depending on use.

7.7.2 Effect of structure size

The nature of bow impact loading implies that larger average pressures will be exerted on smaller regions of the structure than on larger regions. Therefore, different load levels would be expected for different types of structure:

- Small sections of bow plating, etc. would need to be designed for the highest pressures which can occur locally.
- Horizontal stiffeners would be subjected to similar pressure levels, since the slam event could occur along most of the length of the stiffener simultaneously.
- Vertical stiffeners and bulwarks would be designed for lower equivalent pressures, since not all of the stiffener would be loaded at the same time (the load is localised or progressive).
- Global structures would be subjected to even lower average pressures as the integrated load at any time includes forces on significant regions of the bow that are not simultaneously loaded.

This effect is illustrated by considering the four sample structures shown in Figure 7-24:

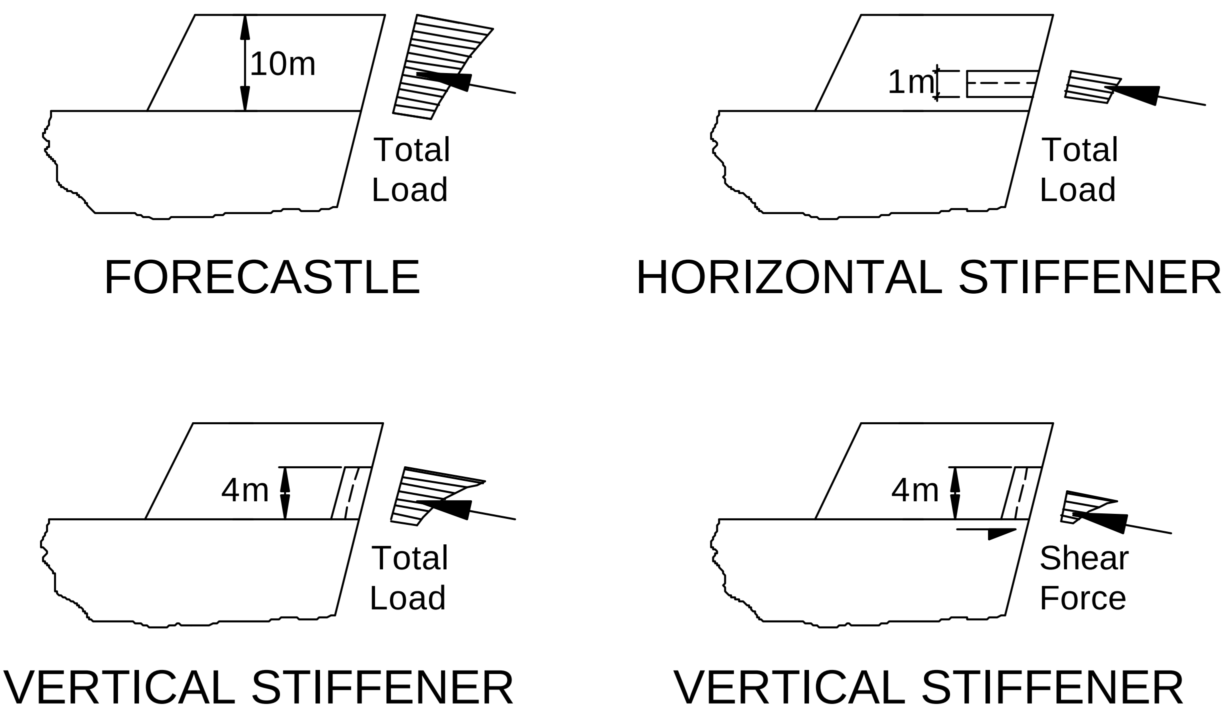


Figure 7-24: Sample bow structures

The last example is intended to show the difference between shear force at one end of a stiffener and overall loading on the stiffener as a whole. Equivalent pressures have been derived for each of these structures for different heights up the bow of the vessel.

These results are presented in non-dimensional form in Figure 7-25:

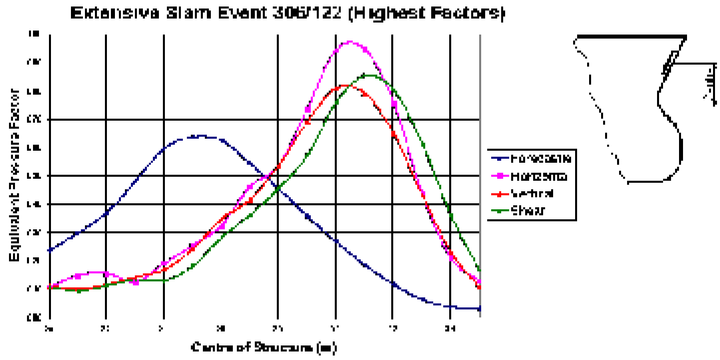


Figure 7-25: Equivalent pressures for four different structures as function of its height on the bow

The following table presents these results numerically, including the dynamic amplification factor that varies significantly depending on the type of structure considered.

Structure	Factor	DAF	Combined
Forecastle	0.41	0.97	0.40
Vertical stiffener - Bending	0.52	1.14	0.61
Vertical stiffener - Shear	0.57	1.14	0.65
Horizontal stiffener	0.80	1.33	1.07

These results clearly indicate the significance of the following:

- The size of the target structure, in particular, its vertical extent, with higher loads being developed on smaller structures.
- The larger magnitude equivalent pressure load for shear compared with bending.
- The effect of shorter rise and fall times for smaller structures exaggerating the increase by greater dynamic amplification.

Hence it is very important in design to use pressure loading that is consistent with the size of structure being designed.

7.7.3 Bow plating

Plating thicknesses at the bow of up to 20 mm are not uncommon, depending on spacing of stiffeners and location around the bow. Spacings between stiffeners of 750 to 1000 mm are typical. Thicknesses tend to reduce to less than this value remote from the centreline of the vessel. The extent to which this reduction is possible depends on the variation in slam loading with angle around the bow, which in turn depends on vessel heading control.
There is no direct data from the tests or the real time measurements to illustrate the behaviour of bow plate subject to slam loading. Bow plate was not directly monitored on BP Schiehallion. To understand plating response, finite element analysis of the plating on BP Schiehallion was performed under the action of slam loads of different magnitude, rise time and movement across the plate. This analysis is contained in the WP3 Technical Report on structural response to wave impact loads.

There is seldom a concern over strength of the hull plating itself, provided that this is adequately supported by stiffeners behind. The analysis supports the conclusion that there are three possible mechanisms for bow plating to carry lateral pressure loads, depending on the magnitude of this load:

- By arch action, due to the curved initial shape of the plate.
- By bending of the plate itself.
- By membrane action, the lateral load being resisted by tension in the deformed plate.

Arch action cannot normally be relied upon for local bow plating, since curvatures of the plate are small and are often exceeded by construction tolerances or displacements once the structure has been subject to initial slams.
Hull plating is expected to be able to carries only part of any significant external slap pressure loading in bending. Although For example, a 17.5 mm plate spanning between stiffeners 900 mm apart (as at Schiehallion) would be able to carry only 14 m head of water prior to the onset of yielding. Localised slam pressures up to 10 times significantly in excess of the above have been observed in tank tests and in reality.
However, considerable additional load may be resisted by the bow plate acting as a membrane, akin to the action of a trampoline. Tensile stresses are created in the plate as it flexes to resist the applied pressure.

This action results in some yielding and permanent set of the deck plate at its supports where bending strains combined with axial strains are large, so enhancing producing the characteristic 'hungry horse' appearance of a structure after such loading. Non-linear finite element analysis included the WP3 Technical Report illustrates how permanent deformations of the plate observed at BP Schiehallion (in regions where the supporting structure did not fail) could have been created.
Membrane action of the plate provides for a very ductile response in the structure. Large deformations can occur, absorbing energy, prior to any failure. Plate thicknesses therefore do not need to be too large for significant capacities to be obtained. Thickness requirements should consider:

- The level of restraint from adjacent structure.
- Dynamic effects.
- Stresses induced in the deck plate due to flexure of the stiffeners behind.

Because of this change from bending to membrane action, the notional Ultimate Strength Ratio for bow plating is very high, even if peak strains are limited to prevent fracture. USR values depend on thickness to span ratios, as noted in the WP3 Technical Report. USR values in excess of 10.0 (calculated relative to first yield in bending) are not uncommon.
It is important that uniform support is provided to the bow plate so that local stress concentrations do not occur where fracture could result. Rat holes, if present, should be relatively small. The presence of welds in the bow plate can also weaken the plating by fracture. It is important that these do not occur too close to the high strains at supports.
Care is also normally needed at corners or edges in plated structures, since membrane action relies upon continuity of the plate to balance membrane stresses in one panel by similar stresses in the next. This is generally not an issue for the bow structure, since the outer shell is continuous, but may be significant for deck houses under green water loading.

The action of deck plate in part bending and part membrane action complicates natural frequency calculations for the plate itself. However, the non-linear transient dynamic finite element analysis of the Schiehallion deck plate in the WP3 Technical Report suggests that the frequency of response of deck plate under expected peak pressure loads is typically
10 times the frequency of the stiffeners supporting it, perhaps in the order of 200 Hz. However, a range of frequency from, say, 50 Hz upwards should be considered to allow for different designs, added mass, local support at the ends of panels, etc.
The same analysis shows that the peak structural response varies with the rise time of the slam load. This is confirmed by the MathCad spreadsheets in the WP3 Technical Report for instantaneous and progressive slam events. These spreadsheets calculate the dynamic response of the bow plate under the action of a typical slam time trace that moves up the bow structure at a specified speed. For an instantaneous slam (very fast movement), the peak response is obtained for a natural period that is approximately equal to twice the rise time of the pulse. For progressive slams, the critical period (for maximum dynamic amplification factor, DAF) is approximately given by $T_{crit} = 2 x (T_{rise} + L/V)$, where L is the characteristic size of the component and V is the speed of the water surface across the structure.
Calculations on bow plating response are included in the WP3 Technical Report.

For both instantaneous and progressive slam events, the maximum DAF is about the same, at 1.6. However, the required natural period for this to occur is very different at 0.025 seconds for the instantaneous case, but 0.11 seconds for a progressive slam travelling at 20 m/s. From above, it is conceivable that the natural period of the plate could be in the region of the former, but it will not be as high as 0.11 seconds. It is therefore concluded that the plate will respond dynamically to an instantaneous slap, but not to a progressive slam event. A suitable dynamic amplification in the first case would be around 1.60.
It is therefore most appropriate to assess the bow plate as a membrane under the action of slam and slap loads, with a suitable DAF if the pressure is instantaneous. However, the bow plate also acts as the forward flange of its supporting scantlings, stiffeners or frame. Stresses due to flexure of that supporting structure should be considered in assessing bow plate strength. This may not be too onerous, since the neutral axis of the combined section is close to the bow plate itself and bending stresses will be relatively small.
Additionally, these stresses will act perpendicular to the direction of the principle membrane stresses and the two need only be combined via some interaction equation, such as the von Mises equivalent stress formulation. The worst case will probably be at mid panel, where compressive stresses from flexure combine with tensile membrane stresses.

7.7.4 Stiffeners

Typical ship construction features longitudinal stiffeners supported by evenly spaced transverse frames spaced at 3 to 5 m c/c. Towards the bow of the structure, this framing pattern is maintained for as long as possible, but the transverse frames becoming truncated as the depth of the hull reduces with rake angle. The complexity of the bow shape often results in irregular framing details between the typically vertical bow stiffeners and the standard hull construction further aft. Angled ‘cant’ frames are not uncommon.

At the very bow itself, transverse frames are impractical and an alternative system is often used.

Vertical stiffeners or longitudinal frames (running up and down the bow) are optimum for impact loads, since such stiffeners span directly between strong points (decks) and are not simultaneously loaded by traditional progressive impact loading. Horizontal stiffeners tend to be loaded more rapidly by impacts progressing up the bow, as illustrated in Section 7.7.2. This results in higher simultaneous loading and greater dynamic response. Design loads on horizontal stiffeners may be as much as 50% greater than for vertical stiffeners.

Further aft, the framing pattern must integrate with the normal hull construction (scantlings and transverse frames). The location at which it is acceptable to adopt more conventional structure depends on vessel heading control and the reduction of slam pressure with the angle away from the wave heading. Data on the response of stiffeners or components of similar size under impact loading comes from several sources:

- Real-time monitoring of the Schiehallion FPSO bow, including strain gauging of the vertical stiffeners.
- Pressure panels on the MARIN model of Schiehallion for SAFE-FLOW.
- Stiffener sized panels on the EPSRC model of the Loch Rannoch.
- Finite element stress analysis presented in the Technical Report.

Data on the response of local structure to slap loading comes from two sources:

- Real-time monitoring of the Schiehallion FPSO, including strain gauging of the vertical stiffeners.
- Pressure panels on the MARIN model of Schiehallion for SAFE-FLOW.

Monitoring results from BP Schiehallion show evidence of significant dynamic response in the most some bow stiffener stress records. Dynamic amplification factors in excess of 2.0 are observed. This observation is supported by the WP3 Technical Report, which shows reasonably significant dynamic amplification of response to any form of locally instantaneous slam. As for the bow plate, the extent of dynamic amplification depends on the ratio of the load rise time to the structural natural period. In the case of Schiehallion, the natural frequency of the stiffener has been taken as 40 Hz (which is within the range expected for partially immersed structures) and the maximum DAF is about 1.6.

Similar calculations have also been performed to derive the response of stiffeners subjected to progressive type slam events. Despite reducing the natural frequency to 20 Hz (the minimum that would be expected with full immersion), dynamic response is significantly less, at 1.08. This shows the relative significance of the simultaneous loading to structural response. The progressive slam event is much more benign - not only is there less dynamic response, but also the peak pressure tends to progress relatively steadily up the bow, as immersion occurs. This does not create large simultaneous loads on the bow structure and equivalent static pressures in the region of 50 to 60% of the peak instantaneous pressures are obtained in this way.

Attempts to use full-scale monitoring results to correlate local pressure readings with response of the bow stiffeners have proved to be difficult (see WP3 Technical Report). Several parameters have been identified that have a bearing on the impact response, including pressure duration, extent of immersion, spatial extent of the slam, etc.

The magnitude of the peak recorded pressure on its own is not necessarily representative of the impact. The difficulty is that again, the pressure sensors are not necessarily located at the elevation of the peak slam response and may therefore give misleading information for localised slams. Further work is required here when a better database of results is available.

In summary, the type of slam loading is fundamental to the response of the bow stiffeners. Progressive slams are relatively benign, but significant response may result if the slam event is simultaneous over a reasonable area of the bow.

It is not necessary to design the structure elastically under this most extreme level of loading produced by these simultaneous slams.
Bow structures adequately designed for more modest events would be expected to have sufficient reserve strength to resist the greater pressures that would result from a simultaneous impact, albeit with some inelastic response and plastic deformation. For the geometry of stiffening on BP Schiehallion, this reserve strength may be quantified as follows:

- The plastic moment capacity of the stiffener plate section is some 1.35 times the elastic modulus, this being the shape function of the section.
- Failure of the stiffener will not occur until the formation of plastic hinges at the centre and ends of the member.

The combined effect of the above suggests that the load at failure would be approximately 1.8 times the load at first yield. This is typical of the Ultimate Strength Ratio (USR) for this type of structure. For this to be achieved, the stiffener should be detailed so that premature shear of the ends, failure of the welds or buckling of the web or flange will not occur. The above USR assumes that the stiffener is continuous over supports. A significant reduction is USR should be considered if the stiffener is simply supported or propped. In such cases, there is less possibility of load redistribution and less redundancy.

Shear USRs would be expected to be significantly less than this value, in the region of 1.2. This highlights the relative importance of shear loading, which is already expected to be higher than bending loading (see Section 7.7.2). The only redundancy is in the redistribution of stress across the web and the effects of strain hardening, etc. It is recommended that shear connections be carefully detailed and attention paid to weld quality and strength in regions where wave impact loads are high.

7.7.5 Supporting structures

It is important that pressure loads on the bow structure are adequately and safely carried back into the main hull structure. Supporting structures should be locally strong enough not to buckle or yield as a result of these principally compressive loads on the forecastle. Such structures include:

- Primary framing supporting stiffeners.
- Deck structures to which stiffeners and frames are connected.
- Bulkheads that support the decks and some stiffeners.

These structures are typically loaded in compression by slam loads, with the result that buckling may occur, as at BP Schiehallion. This effect may be exaggerated by bow flare and rake, which causes these compressive loads to be applied eccentrically to the plane of support. Simultaneous high pressures over the parts of stiffeners local to such structure would be the worst design condition.
Significant dynamic response of the supporting structure is unlikely (but see 7.7.6, below, for the global dynamics of forecastle structures). However, Ultimate Strength Ratios are expected to be relatively small due to the failure mode of such structures being buckling. USRs in the region of 1.4 should be considered.

The bow space is normally reserved for non-oil storage or ballast, providing a twin hull to prevent oil spillage in the event of collision. There is normally a transverse bulkhead just aft of the bow section that marks the start of the oil storage tanks. On converted trading tankers, this is typically a robust collision bulkhead. Additionally, there are typically normally longitudinal bulkheads on centreline and to port and starboard of the cargo tanks. Advantage may be taken from this form of construction in the selection of probabilities of failure for reliability calculations.

7.7.6 Forecastle structures global loading

Forecastle structures above the main deck tend to present particular problems in transmitting slam loads back into the main structure, due to their height above the main deck. Both the horizontal shear components of slamming load and overturning of the forecastle must be resisted. Side shells and longitudinal bulkheads both typically contribute to the necessary strength. It is important that any structure above the main deck be adequately ‘backed-up’ by supporting steelwork below.

It is clear from the results of tests and calculations that overall loading on the bow structure occurs over a much longer period than for local structures. This is as expected, since the load takes a finite time to be applied to the overall bow structure. MARIN results show that the peak load may occur up to a second after first immergence. Glasgow University tests, which considered far more severe waves, produced far more rapid loading, particularly for instantaneous slaps. However, even here, there are significant delays as the load progresses up and around the bow structure.

Despite this increase in the duration of loading, dynamic effects can still be significant, as the natural period of the forecastle is also significantly greater than the bow plate and stiffeners previously considered. By comparison, natural frequencies of around 100-200 Hz, 25-50 Hz and 4-9 Hz may be expected for bow plate, stiffeners and forecastle structures at Schiehallion.

A comparison of modelled and actual periods has been performed for the MARIN tests and their effect on a typical slam has been assessed. Spreadsheets in the WP3 Technical Report present the results of a simple single-degree of freedom system. The dynamic amplification for the as-modelled condition is 12%, which very much agrees with observations from the response time traces. With the actual mass used in the spreadsheets, the DAF reduces to 3%. The bow structure used of the numerical calculations in Section 7.7.2 has a DAF marginally less than 1.0.

Loading is expected to occur far more quickly for instantaneous slap events identified by Glasgow University for very extreme (steep faced) waves, see Barltrop and Xu (2004). Calculations are included in the WP3 Technical Report to attempt to simulate the observed response of the forecastle structure for a typical instantaneous loading event. A very different form of response is obtained because in this case, since the load frequency is higher than the natural frequency. The result is that the structure does not have time to respond to the slap event, and peak response is reduced.

It is concluded that there is unlikely to be significant dynamic response for robust forecastle structures under localised or progressive slap events. The reason for this is clearly that typical peak slam loading does not occur simultaneously across the structure, neither up the bow nor around it. For instantaneous slaps, it is possible for the slam load duration to be less than the natural period of the structure, resulting in dynamic amplification less than 1.0. As with other types of structure, the type of loading, localised, progressive or instantaneous, is significant for determining response. Specific checks on natural periods should be provided where the forecastle structure is dissimilar in form to Schiehallion.

Ultimate Strength Ratios (USRs) are difficult to determine for global structures such as forecastles, depending on many factors such as:

- The number of load paths back into the supporting structure.
- The possibility of progressive collapse (failure of one load path causing a knock-on effects).
- The definition of first yield in the structure (local or global components).
- The reserve strength of components against buckling.

It is recommended that USRs for forecastles be based on the USR of the primary load transmitting members (e.g., the side shell or bulkheads in buckling). Consideration may be given to redundancy where there are multiple load paths back into the supporting structure, provided that progressive collapse of these load paths is not likely.

7.7.7 Bulwarks

Bulwarks are a continuation of the hull bow structure above the upper deck and act to minimise the possibility of green water on the deck. Vertical framing of the bow structure below is an advantage in providing continuity of support to the bulwark structure, which effectively cantilevers upwards from the top deck. Further around the bow, it becomes difficult to provide continuity with structure below and support details tend to be varied and complex. Particular attention should be given to providing such continuity in design.

Results of the JIP 'FPSO Green water loading' concluded that vertical were the optimum configuration for bulwarks, to minimise their effect on vessel motions. The loading on a vertical bulwark structure would also be expected less severe than on raked or flared bow plating. During immersion, water forced upwards by a slam would be expected to try to follow this rake angle and not impinge on a vertical bulwark.

There is no data available at this time to determine bulwark loads or to support the above assumption that design loads should be less on vertical bulwarks. Consideration should be given to including bulwark loads in future studies. At present, it is recommended that the bulwarks (even if vertical) should be designed on the same basis as the remainder of the bow structure.

Bulwarks are effectively cantilever structures. This has the following effects:

- Natural periods would be longer than for simply supported or encastré beams of similar size and length, but because of their reduced span, dynamic amplification is not expected to be worse than for stiffeners (a DAF of 1.6).
- There is little of no redundancy in the load carrying mechanism, with the result that USRs are expected to be relatively small, in the region of 1.35 if the structure is well detailed. Shear USRs would be expected to be even less, at 1.2.

Bulwark structures may be designed to be damaged if very extreme events occur. This failure mechanism should be to collapse the bulwark against the deck, rather than to shear it off so that it becomes a projectile. Detailing of these structures should reflect this aim.

7.7.8 Hull girder loads

Hull girder bending due to wave impact loads was evaluated as part of the EPSRC study for the Loch Rannoch model and has been reported in Barltrop and Xu (2004). Both hull girder bending due to bottom slamming and due to wave impact on the bow has been considered. Typical results are illustrated in Figure 7-26, below:

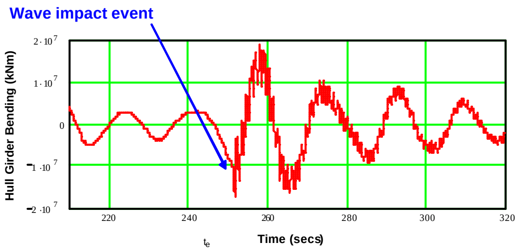


Figure 7-26: Hull girder bending during wave impact event

The measured bottom slamming forces were an order of magnitude smaller than the bow impact forces. This is a consequence of the impact velocities being about 1/3 of the celerity and the forces being proportional to impact velocity squared times the slammed area. If the areas are about the same this leads to a force of about $(1/3)^2$ or about 10%. The bottom slamming effect on hull girder bending moment is important because of the long lever arm of the vertical forces but the high horizontal forces at the much smaller vertical lever arm also produce significant vibration in the hull girder.

The magnitude of this hull girder bending due to wave impact for the Loch Rannoch vessel was about 25% of the design hull girder bending moment, which is significant. It is also apparent that this load can occur simultaneously with hull girder sagging and hogging moments, for the following reasons:

- Damping of the dynamic whipping effect appears to be small (see above), with the result that the hull girder vibrates significantly through a couple of cycles of hogging and sagging.
- The hull girder natural period is short relative to the wave period, with the result that dynamic response is almost directly additive to wave frequency response.
- Significant wave impact can occur similar sea states to maximum hull girder bending.
- Individual steep faced waves that cause wave impact, whilst not necessarily producing worst wave frequency hull girder bending, can cause very significant moments.

In view of the above, it is recommended that hull girders of FPSOs be checked for the combined effect of wave frequency and dynamic wave impact bending. For the latter, maximum wave impact loading on the forecastle structure should be determined and used to calculate a bending moment at midships, taking into account the bow rake angle. No reduction due to damping should be considered.

7.8 Fatigue and fracture

7.8.1 General

Fatigue crack growth is conceivable within structures subject to wave impact for two reasons:

- The bow structure, particularly near the water surface, is subjected to repeated stress cycles due to pitch and heave immersion below the water surface. This results in pressure induced stresses as a result of hydrostatic head and diffraction/radiation/hydrodynamic effects.
- In addition to simple immersion effects, the bow is subject to repeated slam loads.

An sample calculation has been performed for the Schiehallion bow structure, based on the following data:

- Pressure and stress monitoring results from the bow of the Schiehallion FPSO.
- History of pressure loading recorded in ‘bins’ over 2 years.
- A relationship between pressures and stresses recorded and checked by FEA.
- A local stress concentration factor of 1.5.
- A F2 detail at a typical weld low down in the bow structure where there are more immersions.

The following Figure 7-27 shows exceedances of pressure at the lowest pressure sensor. The blue line indicates what may be classed as simple immersions. These are relatively frequent and tend towards the maximum static head that can occur. The red line indicates pressures that are too high to be classed as simple static heads, and are therefore representative of impact events (in Barltrop and Xu (2004), there is evidence that at least some of this increase in pressure may not be due to impact loads, but may be caused by increases in hydrostatic/hydrodynamic pressure in steep faced waves).

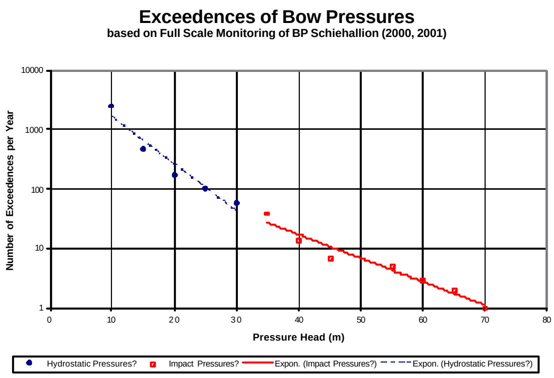


Figure 7-27: Bow pressure probability from Schiehallion monitoring

As noted above, a relationship between pressure and stress range has been determined based on observed simultaneous records of each. For the selected location low in the bow structure, the stress range in MPa is equal to 8.5 times the head in metres. The following table then shows the stress ranges and resultant damage for the selected location, based on the above fatigue data:

Pressure/Stress Occurrence Data

Head (m)	Occurrences	Exceedences	Stress (MPa)	Cycles to Failure	Damage
10	2000	2471	85.0	206859	0.009668
15	300	471	127.5	61291	0.004895
20	70	171	170.0	25857	0.002707
25	45	101	212.5	13239	0.003399
30	17	56	255.0	7661	0.002219
35	25	39	297.5	4825	0.005182
40	7	14	340.0	3232	0.002166
45	2	7	382.5	2270	0.000881
55	2	5	467.5	1243	0.001609
60	1	3	510.0	958	0.001044
65	1	2	552.5	753	0.001328
70	1	1	595.0	603	0.001658
				Total	**0.036755**
Based on F2 curve with SCF of 1.5				**Life (years)**	**27.2**

The resultant life is marginal. Although it exceeds the intended platform life, it does not do so by sufficient margin. A factor of safety of three would normally be required. The following Figure 7-28 shows that slam loading (above about 30 m head) accounts for at least 30% of the cumulative fatigue damage at this critical location.

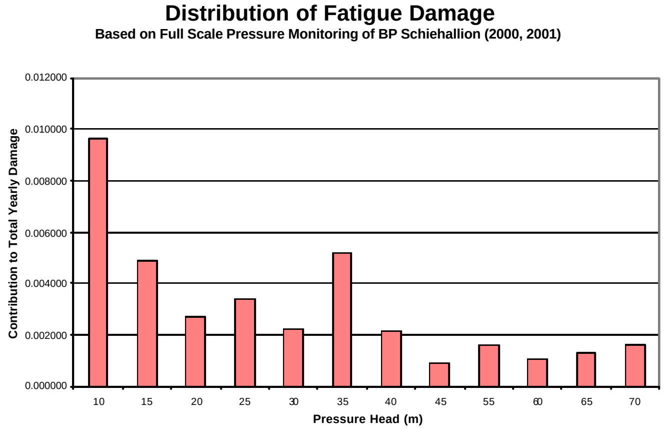


Figure 7-28: Hull girder bending during wave impact event

The above results indicate that wave impact can have a reasonably significant effect on fatigue damage to bow structures. The very high stress ranges that result, whilst relatively few in number, can contribute significantly to overall damage.

It is therefore recommended that checks be performed on bow structure for fatigue damage. In the absence of other data and on the basis of the above calculations, it is recommended that these calculations be based on simple hydrostatic/hydrodynamic immersions. The resultant fatigue damage should then be doubled to allow for wave impact effects. Where this indicates fatigue lives that do not have an acceptable margin over target life, further checks should be performed or the structural detailing improved.

7.8.2 Fracture

Fracture is a possible cause of structural failure that may result from:

- excessive strains in the material.
- poor fabrication of the structure resulting in defects at which fracture can occur at reduced strain.
- the growth of fatigue cracks in the structure, again causing fracture at reduced strain.
- inappropriate use of brittle materials, particularly in cold weather conditions.

The very large magnitude stresses and inelastic strains that result from bow impact loads can cause fracture when even relatively small defects are present. In the following Figure 7-29, which considers stresses up to yield, the permissible crack size is only 2 mm for a loading that is applied once only. The permissible defect size reduces rapidly with number of wave impacts to which the structure is subjected.

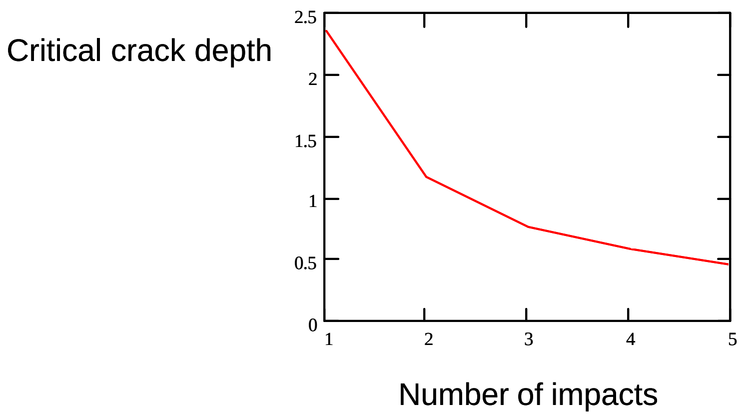


Figure 7-29: Effect of number of impacts on critical defect size

Defect sizes up to 3 mm are not reliably detectable in welds, with the result that fracture is a potential concern. It is recommended that areas of repeated wave impact be regularly inspected for defects and fatigue crack growth, which could severely weaken the structure.

7.8.3 Fabrication quality

The fabrication of complex bow shapes is often difficult and this may result in poor fit-up of components in an area where high quality is required. Standard shipyard details may not be sufficient where high impulsive loads are predicted. Typical fatigue details are not that important. The structure should also demonstrate ductility so that in the event of very high slam loads, plastic deformation, rather than fracture, will occur.

8 WATER ENTRY PROBLEMS

8.1 Introduction

During the SAFE-FLOW project the main focus was on green water loading (making mainly use of test results from the previous JIP 'FPSO Green water loading') and wave impact loading on the bow.
This chapter gives some additional information of the related subjects of:

- bow flare slamming, based on (limited) experiments in the JIP 'FPSO Green water loading'
- loads on external turrets (chain tables), based on a limited model test series in the SAFE-FLOW project.

8.2 Bow flare slamming

8.2.1 Measurements

In the JIP 'FPSO Green Water Loading' some work was carried out on the subject of bow flare slamming. Tests were carried out with a large bow flare angle of 50 degrees with pressure panels in the bow flare section, see Figure 8-1.

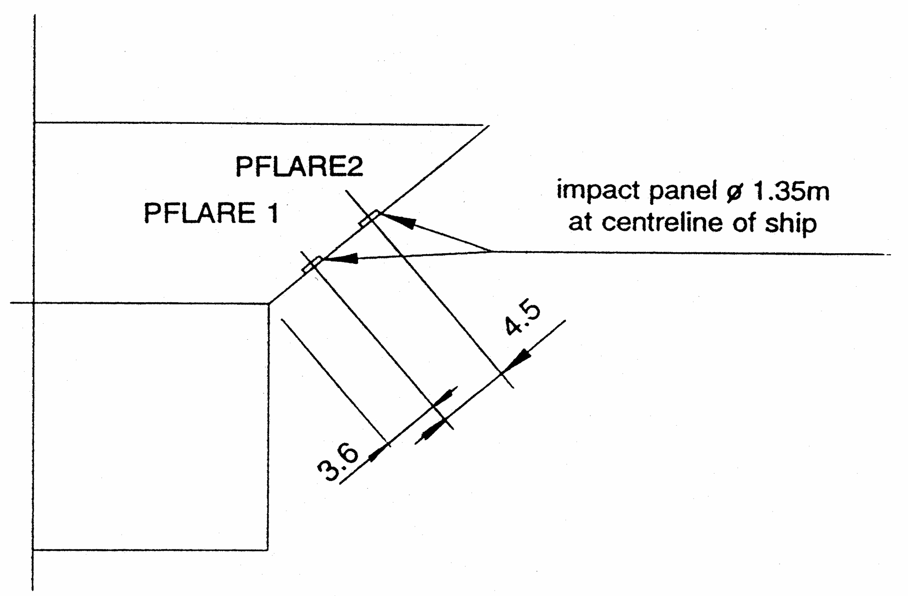


Figure 8-1: Bow model with pressure panels to measure bow flare slamming

A limited number of regular and irregular wave tests was performed with this set-up. In Figure 8-2 the typical time traces of the relative motions (R2), the relative velocity (R2 VEL) and the impacts on the two pressure panels is shown.

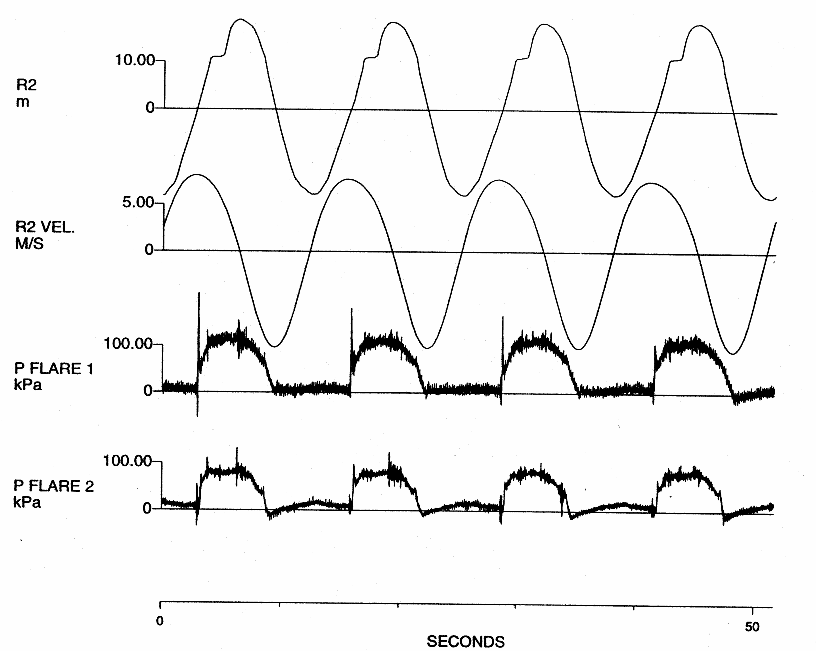


Figure 8-2: Typical time traces of the relative motions (R2), relative velocities (R2 VEL) and impacts on the pressure panels in a regular wave

At the moment that the bow flare is entering the water a peak bow flare impact pres-sure is shown below. After that the bow flare is submerged in the water and the pres-sure is mainly due to the quasi-static water pressure in the wave. In irregular waves a maxi-mum peak pressure of approximately 500 kPa was found. Figure 8-3 shows the histogram of the pressures (PFLARE 1) in Figure 8-3.

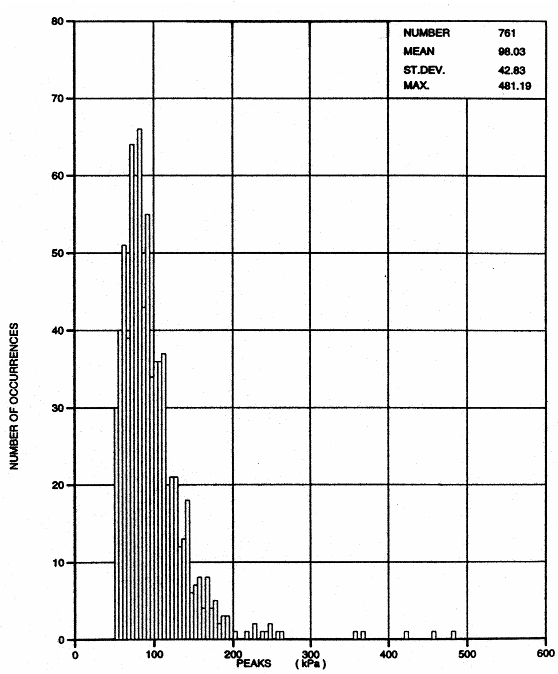


Figure 8-3: Distribution of measured pressures on the bow flare in irregular wave

8.2.2 First estimation approach design guidance

This information confirmed that bow flare slamming on bows with large flare angles can be a problem. But from the limited number of tests in the JIP no definite method for the estimation of bow flare slamming can be developed. Some guidance in the determination of peak pressures on bow flares can be found in the extensive work on wedge entries, see for instance Zhao and Faltinsen and Zhao, Faltinsen and Aarsnes. Because of the symmetry assumptions in this type of investigations, the results can also be used to estimate the local flow around the bow flare of an F(P)SO on a further vertical bow, see Figure 8-4.

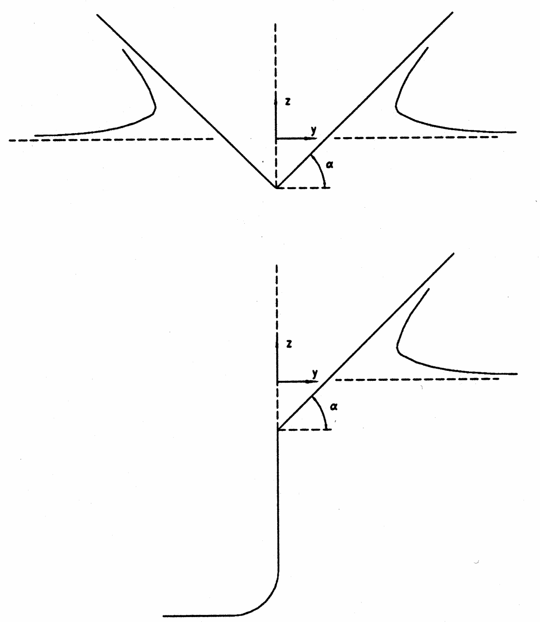


Figure 8-4: Due to the symmetry assumptions, wedge entry results can be used to estimate bow flare slamming

The bow flare angle γ can be determined from the dead rise angle α according to (in degrees):

$$\gamma = 90 - \alpha$$

Zhao, Faltinsen and Aarsnes applied different methods to come to a solution for the pressure on the wedge, including similarity flow and a boundary element method. Their results are presented as an impact coefficient Ci according to:

$$C_i = \frac{p - p_a}{0.5\,\rho\,V^2}$$

V is the entrance velocity and pa is the atmospheric pressure. The dead rise angle does not only influence the pressure coefficient, but also the position of the maximum pressure on the bow flare. For small dead rise angles (large bow flare angles) the pres-sure peak at the higher part of the flare, for large dead rise angles the peak pressure occurs much lower, see the examples in Figure 8-5 for dead rise angles of 30 degrees (left, bow flare 60 degrees) and 60 degrees (right, bow flare 30 degrees), from Zhao and Faltinsen.

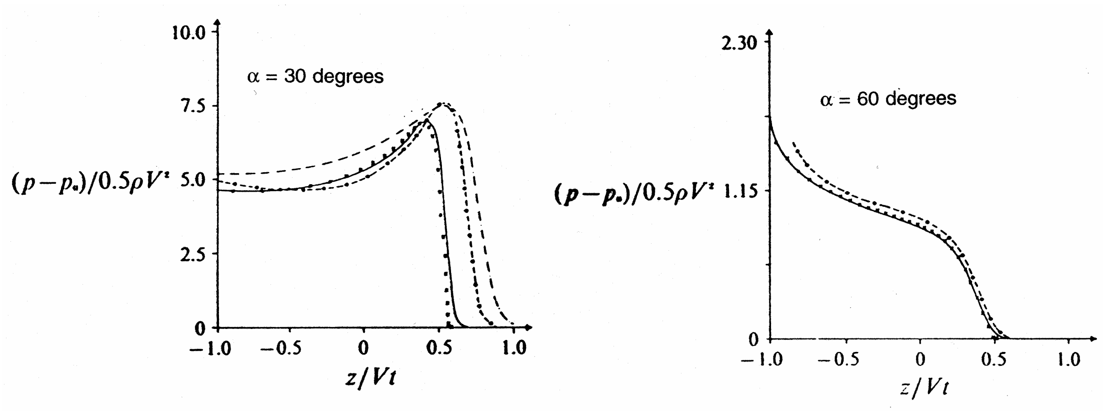


Figure 8-5: Position of the maximum pressures for dead rise angles of 30 degrees (left, bow flare 60 degrees) and 60 degrees (right, bow flare 30 degrees)

Based on the results presented by these authors, it is possible to estimate a maximum bow flare impact pressure, using the relative velocity as the input. The RAO of this relative wave velocity can be found from the RAO of the relative wave motion according to:

$$H_{\dot{r}}(\omega) = H_r(\omega) \, . \, \omega$$

From the results presented in Zhao and Faltinsen now the maximum pressure coefficients C_{max} can be estimated as function of the dead rise or bow flare angle. This is done in Figure 8-6, where it should be noted that the position of the maximum pressure changes with the flare angle.

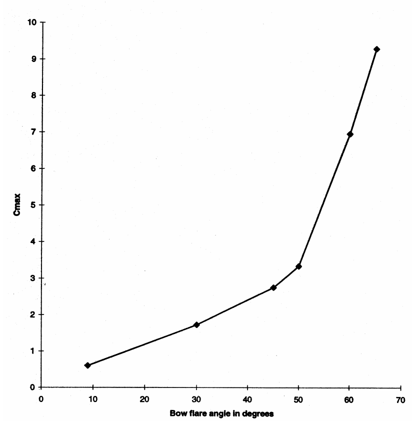


Figure 8-6: Maximum pressure coefficient as function of the bow flare angle

Based on linear theory the relative wave velocity can be determined in either a regular or irregular waves (spectral analysis and determination of MPM value). If we assume that the bow flare enters the wave surface with the bow flare angle, this results in the following expression for the maximum bow flare impact pressure:

$$p = 0.5 \, C_{max} \, \rho \, \dot{r}^2$$

If we apply this relation to the time trace at the beginning of the section (relative wave velocity 8 m/s, C_{max} = 3.4 and ρ = 1.025 t/m^3), we find a maximum estimated pres-sure of 111 kPa, which is slightly lower than the observed pressure peak. The difference might be due to the effect of the pitch angle (5 degrees) and the local wave slope on the impact coefficient.
Due to the fast increase of C_{max} above the flare angle of 50 degrees this pitch angle and wave slope can effect the bow flare impacts significantly. It is therefore proposed to increase the bow flare virtually to account for this type of effects.

8.3 Loads on external turrets (chain tables)

8.3.1 Measurements

During the SAFE-FLOW project a limited model test series was carried out to determine the loads on an external turret (chain table). Figure 8-7 shows the test set-up used (more details can be found in the WP2 Technical Reports).

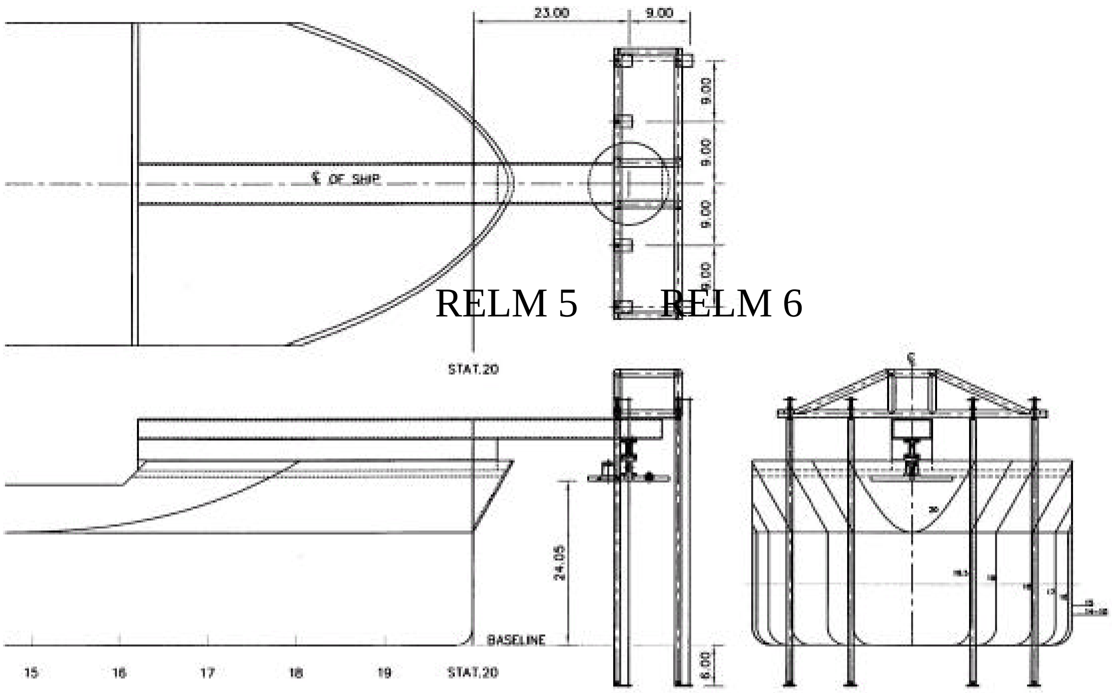


Figure 8-7: Model test setup for measurement of external turret slamming

Measurements were carried out on a flat turret (see Figure 8-8, left) and a conical turret with top angle of 140 degrees (see Figure 8-8, right). The dead rise angle of the conical turret is consequently 20 degrees.

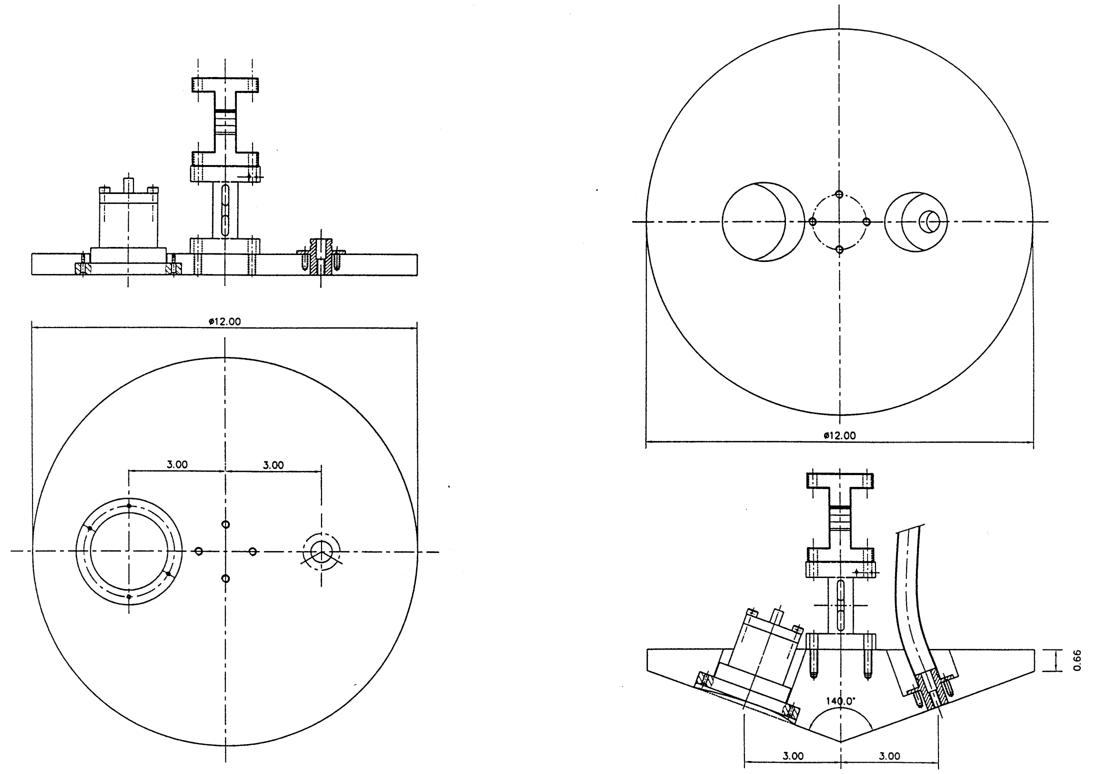


Figure 8-8: Measurements were carried out on a flat turret (left) and a conical turret with top angle of 140 degrees (right)

Three loads were measured on the turret:

- Total vertical load on the turret (FZ TUR).
- Mean pressure over a pressure panel (P PAN).
- Local pressure on pressure transducer (P SLAM).

The airgap between the still water level and the bottom of the flat turret was 6 m.
Figure 8-9 shows a zoomed view of a number of the turret impacts. Beside the normal measurements, also an estimate was made of the wave angle at the moment of impact, based on the difference in relative wave motions at positions 5 and 6 (signal 'WAVE ANGLE'). Further the relative wave velocity was determined by taking the time derivative of the relative wave motion signal (RELM 6 VEL).

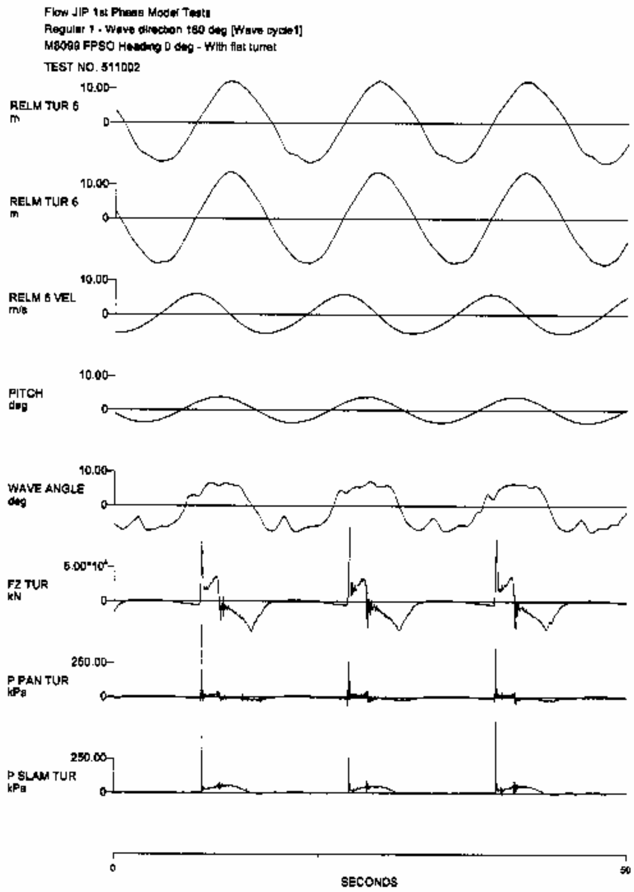


Figure 8-9: Zoomed view of a number impacts on a flat turret

The following can be observed:

- The impact occurs as expected at the moment that the airgap level is exceeded by the relative wave motions.
- Because this airgap is small, the relative wave velocity is still large at the moment. The pitch angle is not at its maximum yet.
- First the force is in the upward direction for about two seconds. At the moment the vessel stops pitching downwards a negative load is observed. From the video it can be observed that when the turret hits the water surface, the water is pushed away. For a certain amount of time the turret goes further down into the water, but the space above the turret is still open. When the turret slows down this area is filled with water, resulting in the negative loading. Afterwards the turret is going upwards and the water starts flowing from the turret.
- The wave angle is between 1.5 and 2 times larger than the pitch angle. The wave angle is much more disturbed that the pitch angle (and it should be noted that this is not the exact situation below the turret, which will be even more disturbed).
- In Figure 8-10 a zoomed view of an impact on the flat turret is compared with an impact on the conical turret. The figure also shows the relative water level measured on the probes around the turret.

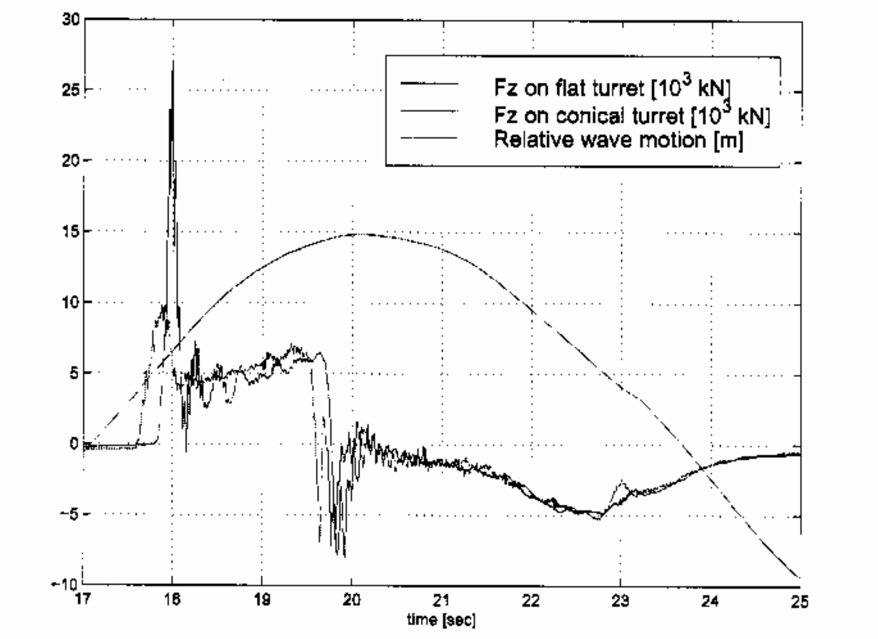


Figure 8-10: Comparison of impact on flat and conical turret for similar relative wave motions around the turret

From Figure 8-10 the following can be observed:

- The rise time of the impact on the flat turret is about 0.15 seconds. Due to its shape, the rise time for the conical turret is longer (about 0.25 s).
- The maximum pressure on the conical turret is significantly reduced compared to the flat turret.
- The pressure build-up for the flat turret starts when the relative motion exceeds 6 m. For the conical turret the pressure build up starts slightly earlier.
- When the turret comes out of the water, the vertical force stays negative for a while. This is a result of an amount of water that stays on top of the turret.
- The impact on an external turret is a combined relative wave motion and relative wave velocity problem. The relative wave motion (r) is defined as the difference between the local vertical vessel motion (z) and the local (disturbed) wave motions (ζ) according to:

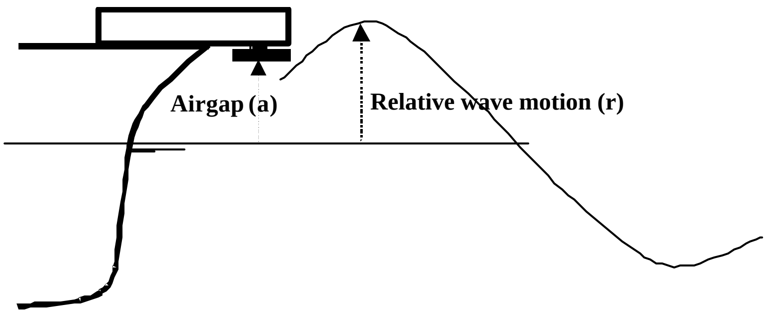


Figure 8-11: Definition of relative wave motion (r) and airgap (a) with respect to the waterline in calm water

When the relative motions exceed the height of the turret above the calm water level (the airgap: a) with a certain velocity, impact loads can occur.
The vessel motions and relative wave motions are generally predicted with 3D linear diffraction analysis. In Figure 8-12 a comparison of the relative wave velocity RAO is given. It is concluded that the comparison between the measurements (lines) and calculations (circles) is good. Consequently linear diffraction analysis can be used for the calculation of the relative wave motions and velocities.

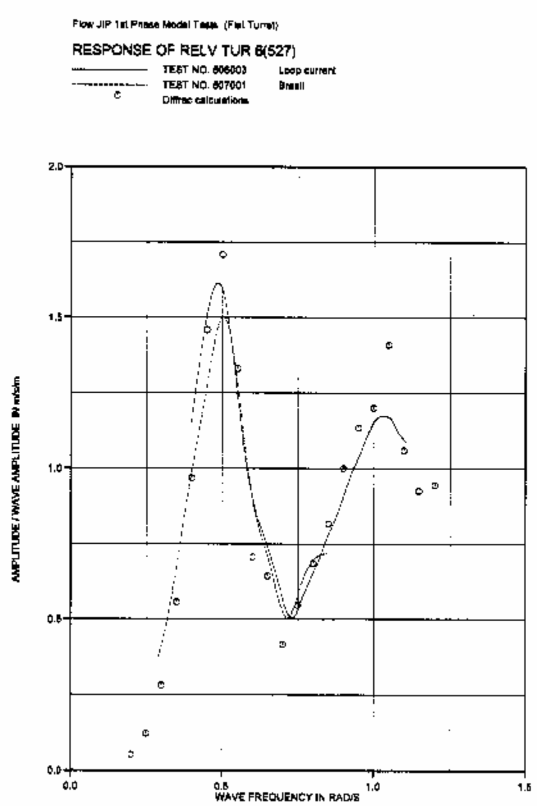


Figure 8-12: The calculated and measured relative wave velocity RAOs show good comparison

Based on the basic physics of water entry (into calm water), a quadratic relation is expected between the impact pressure and the relative velocity:

$$p = C \cdot \frac{1}{2} \cdot \rho \cdot \dot{r}^2$$

C is the impact coefficient, depending on the shape of the body entering the fluid (or the shape of the water surface into which the structure is entering). This relation was checked for the regular wave tests as well as for the irregular wave tests in the Loop current condition (H_s = 6.1 m, T_p = 11 s) and Brasil condition (H_s = 7.8 m, T_p = 15 s). The relative wave motions at position 6 were used to derived the relative wave motions and relative wave velocity (parallel to the position of the actual impact at the centreline of the vessel).

Figures 8-13 and 8-14 show the relation between the average pressure over the turret (total load divided by turret area) for the flat turret (Figure 8-13) and conical turret (Figure 8-14) as function of the relative wave velocity at the moment of impact (submergence velocity).

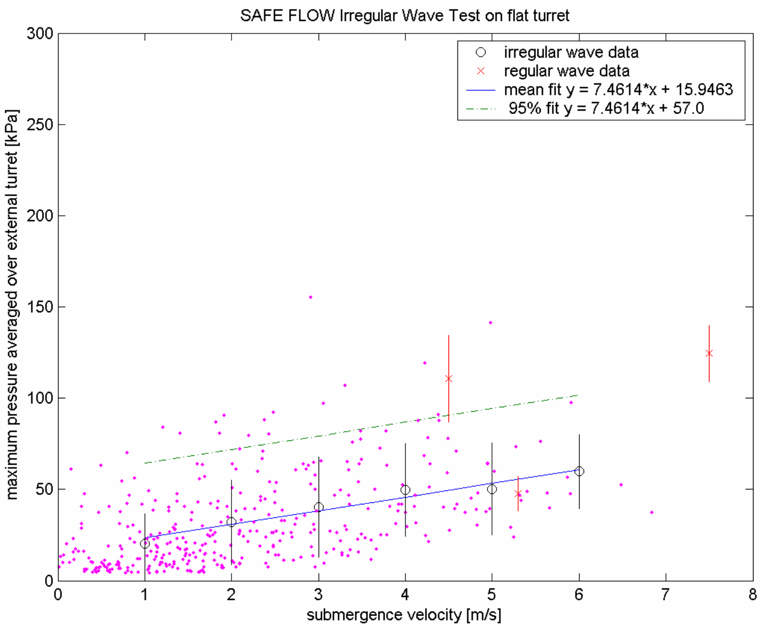


Figure 8-13: Average pressure on the flat turret as function of the relative wave velocity at the moment of impact (submergence velocity)

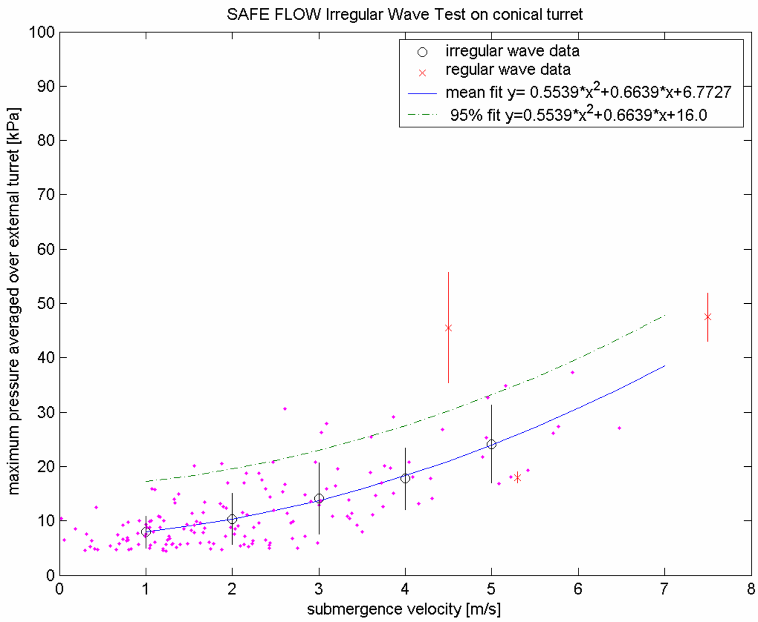


Figure 8-14: Average pressure on the conical turret as function of the relative wave velocity at the moment of impact (submergence velocity)

As was observed for the regular wave tests (also shown as red x's in the figure), there is a considerable scatter of the loads as function of the relative wave velocity at the moment of impact. Further the expected quadratic relation between velocity and impact load is not directly evident from the measurements.

The circles in the figures give the mean load per velocity bin in the plot, the vertical lines through these circles give the +/- standard deviation in that bin. The blue solid line is a linear fit through the circles with mean values. The green dotted line is a line parallel to the mean line, covering 95% of the measurement points (95% reliability line). Something similar, with even more pronounced scatter, was found when we study the maximum local pressures on both turrets. Taking into account the local wave angle (based on the difference in relative wave motions at RELM 5 and RELM 6) did not improve the insight in the relation between relative wave velocity and impact loading.

It is presently assumed that the impact loading process on a structure that is submerging into the water as a result of ship motions combined with wave action, is influenced by more complex effects than only a clear momentum slam in calm water. The main effect seems to be the submergence of the structure in waves that are already heavily disturbed by the presence of the floater and by previous submergences in the wave group. The local dead rise angle during this submergence has a large influence on the final impact loads, especially close to the zero dead rise angle (flat plate). This is shown in Figure 8-15 from Lloyd (1987), presenting the impact coefficient C as function of the dead rise angle.

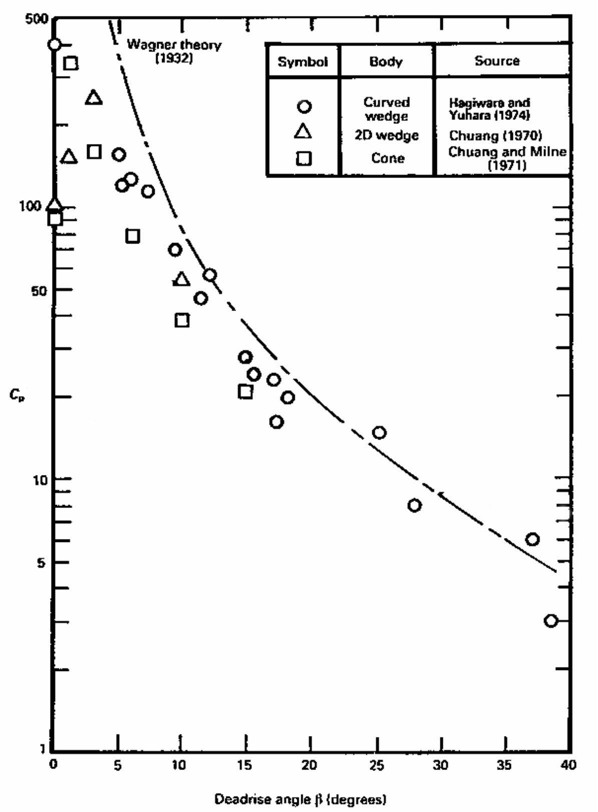


Figure 8-15: Impact coefficient as function of the dead rise angle

8.3.2 First estimation approach design guidance

The (limited) model test results available at the moment do not provide final insight into the loading process of external turret type structures. A clear quadratic relation between submergence velocity and impact load has not been identified. This does not say, however, that the model test results cannot be used for first design estimates. Below a recommendation for such estimates is given, taking into account the findings of the present model tests as good as possible.

Assumptions:

- Although there is no clear quadratic relation between submergence velocity and impact load, the submergence velocity at the moment that the relative wave motions reach the turret is considered to be the main parameter in the loading process.
- In an irregular wave the time trace around the maximum relative wave motion can be considered as a local regular wave signal.
- The maximum submergence velocity at the position of the turret occurs in the same wave cycles as the maximum relative wave motion.

In that case the following procedure can be followed, see also the figure below.

1. The maximum relative wave motion (rmax) in a wave spectrum is determined based on a linear RAO of the relative wave motions and spectral analysis

2. Based on the assumption of a narrow banded linear motion response to Gaussian distributed waves, the Rayleigh distribution applies to the probability of exceedance P of a certain value R of a peak or trough of the relative wave motions in an irregular sea state:

   $$P(r>R) = \exp\left[-\frac{R^2}{2s^2}\right]$$

   s is the standard deviation. Based on these assumptions the Most Probable Maximum (MPM) value of R can be determined. For this value RMPM the following relation applies, with N as the number of maxima in the considered time period:

   $$P(r>R_{MPM}) = \frac{1}{N}$$

   The MPM value under these assumptions can be expressed as:

   $$R_{MPM} = s\,.\sqrt{2\ln N}$$

3. The relative wave motion around this maximum is described as:

   $$r = R_{MPM} \cdot \cos(\omega \cdot t)$$

   Assuming a representative period (and related frequency ?) of the relative wave motion (mean period in the relative motion response spectrum) at the moment of its maximum, the relative velocity can be described as:

   $$\dot{r} = -\omega \cdot R_{MPM} \cdot \sin(\omega \cdot t)$$

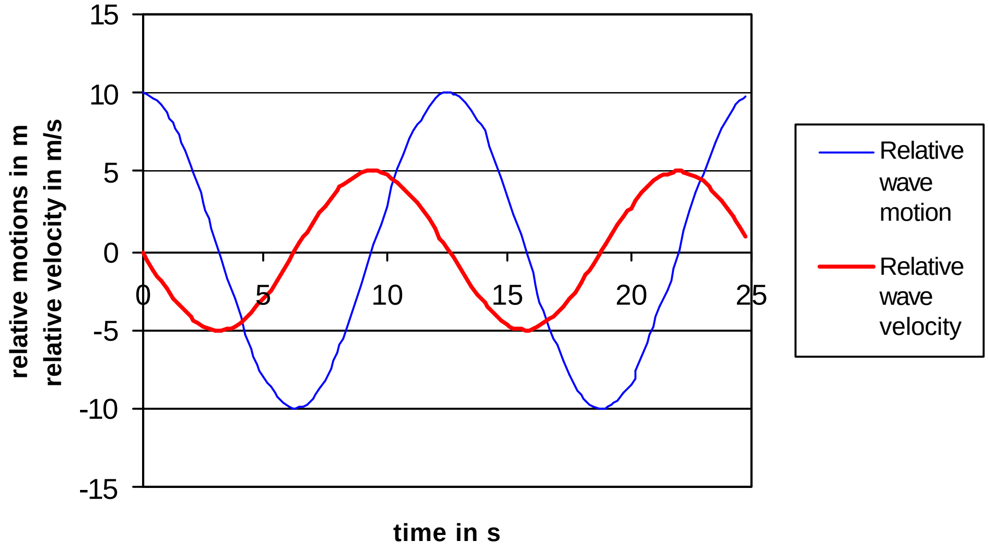


Figure 8-16 Relative wave motion and velocity

Based on these assumptions we can estimate the submergence velocity at the moment the airgap (a) is exceeded as follows, with t_i as the moment of submergence/impact:

$$t_i = \frac{1}{\omega} \arccos\left(\frac{a}{R_{MPM}}\right)$$

$$\dot{r}_i = -\omega \cdot R_{MPM} \cdot \sin(\omega \cdot t_i)$$

As one would expect, this relation shows that the maximum submergence velocity occurs with zero airgap (maximum relative velocity) and that no impact will occur when the wave just reaches the turret (zero relative velocity).

5. Based on these relations the maximum submergence velocity is derived. This can be used to estimate the maximum expected impact load according to:

<u>Flat turret</u>

Maximum pressure averaged over external turret (mean):

$$p_{average,mean} = 7.4614 \cdot \dot{r} + 16.0$$

Maximum pressure averaged over external turret (95% reliability):

$$p_{average,95\%} = 7.4614 \cdot \dot{r} + 57.0$$

Maximum local pressure (mean):

$$p_{local,mean} = 54.1 \cdot \dot{r} + 176.5$$

Maximum local pressure (95% reliability):

$$p_{local,95\%} = 54.1 \cdot \dot{r} + 540$$

<u>Conical turret</u>

Maximum pressure averaged over external turret (mean):

$$p_{average,mean} = 0.5539 \cdot \dot{r}^2 + 0.6639 \cdot \dot{r} + 6.77$$

Maximum pressure averaged over external turret (95% reliability):

$$p_{average,95\%} = 0.5539 \cdot \dot{r}^2 + 0.6639 \cdot \dot{r} + 16.0$$

Maximum local pressure (mean):

$$p_{local,mean} = 2.5788 \cdot \dot{r}^2 - 2.2622 \cdot \dot{r} + 133$$

Maximum local pressure (95% reliability):

$$p_{local,95\%} = 2.5788 \cdot \dot{r}^2 - 2.2622 \cdot \dot{r} + 220$$

9 COMFLOW DEVELOPMENT

9.1 Introduction

A separate part of the SAFE-FLOW project was dedicated to the modelling of water impact problems with Computational Fluid Dynamics (CFD) in the development of the ComFLOW program. This Chapter gives an overview of the developments within the SAFE-FLOW project to indicate how ComFLOW might be used in future design evaluations.

ComFLOW is an improved Volume of Fluid (iVOF) program. The program, formerly called 'Comflo', has originally been developed by the University of Groningen/RuG (Prof.dr. Arthur Veldman) to study the sloshing of liquid fuel in satellites. This micro-gravity environment requires a very accurate and robust description of the free surface. Coupled dynamics between the sloshing fluid and the satellite were investigated as well.

At the start of the SAFE-FLOW project the following functionalities were available in Comflo:

- Calculation of the fluid motion by solving the incompressible Navier-Stokes equations.
- One type of fluid flow is considered, with a void where no fluid is present. This excludes multi-phase flows.
- Possibility to model an arbitrary number of fixed objects in the fluid. The objects are defined piecewise linearly.
- Options to use no-slip or free-slip boundary conditions at the solid boundaries. At the free surface continuity of tangential and normal stresses (including capillary effects) is prescribed. Inflow and outflow boundary conditions for fluid velocities and/or pressures can be defined.
- The fluid simulations are carried out on a Cartesian grid with user-defined stretching. The Cartesian grid is fixed in the domain. When the domain is moving a virtual body force is added to the forcing term in the Navier-Stokes equations. The fluid motions are thus solved in a domain-fixed co-ordinate system.
- To distinguish between the different characters of grid cells, the cells are labelled. The Navier-Stokes equations are discretised and solved in cells that contain fluid. The free-surface displacement is described by the Volume of Fluid method with a local height function.
- During the simulation, forces exerted by the fluid on the submerged objects can be calculated.

During SAFE-FLOW a large step was made in the further development and validation of the program for ship and offshore applications, resulting in the 'ComFLOW' program. In the SAFE-FLOW project the following aspects were developed in ComFLOW:

- Implementation of a generalised geometry description such that the geometry can be defined by an arbitrary finite-element description.
- Implementation of the generation of waves in ComFLOW. This has been accomplished by specifying fluid velocities at the inflow boundary of the fluid domain. The fluid velocities are obtained from potential flow. Linear waves (Airy waves) and fifth-order waves (Stokes waves) have been implemented. The linear waves have been stretched towards the actual position of the free surface.
- The generation of an arbitrary wave pattern (wave spectrum) has been implemented.

- Implementation of several absorbing boundary conditions at the outflow boundaries. First, a Neuman condition has been implemented (specifying the fluid velocities at the outflow boundary based on potential flow, similar to the inflow boundary). Second, a numerical beach (damping zone) has been implemented.
- Improved numerical model to prevent non-physical pressure spikes.
- Possibility to force prescribed movement of bodies in three dimensions.
- The possibility to use the velocity field from a (de-coupled) linear diffraction theory. Linear diffraction theory is used to compute body motions and fluid velocities which are then used in ComFLOW to prescribe the body motions and fluid velocities on the inflow and outflow boundaries.
- An off-line coupling with a structural-analysis code has been established to compute the response of a structure to high peak loads.
- The post-processing facilities have been improved. An offline visualisation program has been made in Matlab. Snapshots of simulations in two dimensions and cross-sections of three-dimensional data can be shown. Snapshots of three-dimensional data can be visualised.

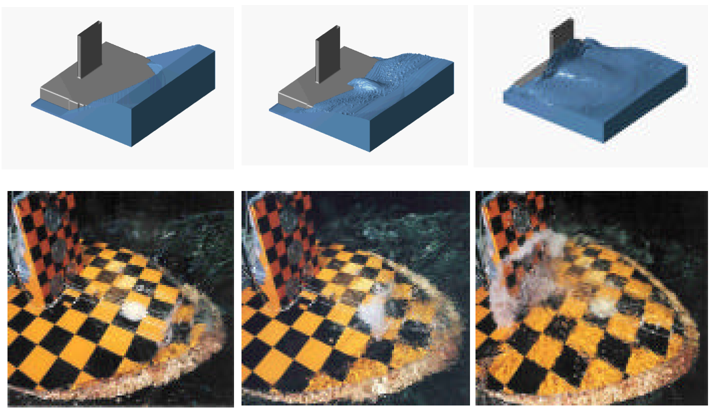


Figure 9-1: Green water ComFLOW simulation and model test

Beside these developments in ComFLOW, a lot of benchmarking and validation was carried out in SAFE-FLOW. During the project also specific validation model tests were carried out. In the SAFE-FLOW project the following validation cases have been studied:

- The propagation of long-crested waves in ComFLOW has been investigated and compared with model basin data.
- The flow of water on the deck of an FPSO with different block type structures on it has been investigated, see Figure 9-1. The sensitivity of the program with respect to mesh refinement and size of the deck structure was tested.
- The wave loading on a fixed object in regular waves has been investigated. For this purpose, model test data on a fixed Spar were used.
- The radiated waves around a moving object in calm water were computed and compared with literature and linear diffraction theory.
- The water entry of a turret was simulated. The impact loads were compared with model test data.
- The wave loads on a rectangular box, located at a short distance above the water surface, were computed, see Figure 9-2. The loads were compared with model test data and other wave-in-deck prediction programs.
- Breaking dam model tests were simulated. The forces on several types of objects (rectangular box, balcony) were compared with the model test data.

- The impact of regular waves on a flared plate was simulated. Impact pressures were compared with measured pressures.
- A comparison was made between the results computed by ComFLOW and by FLOW-3D, a commercial CFD package. As a test case, the wave impact on a box located at a short distance above water was used. A fair agreement between the two packages was found.

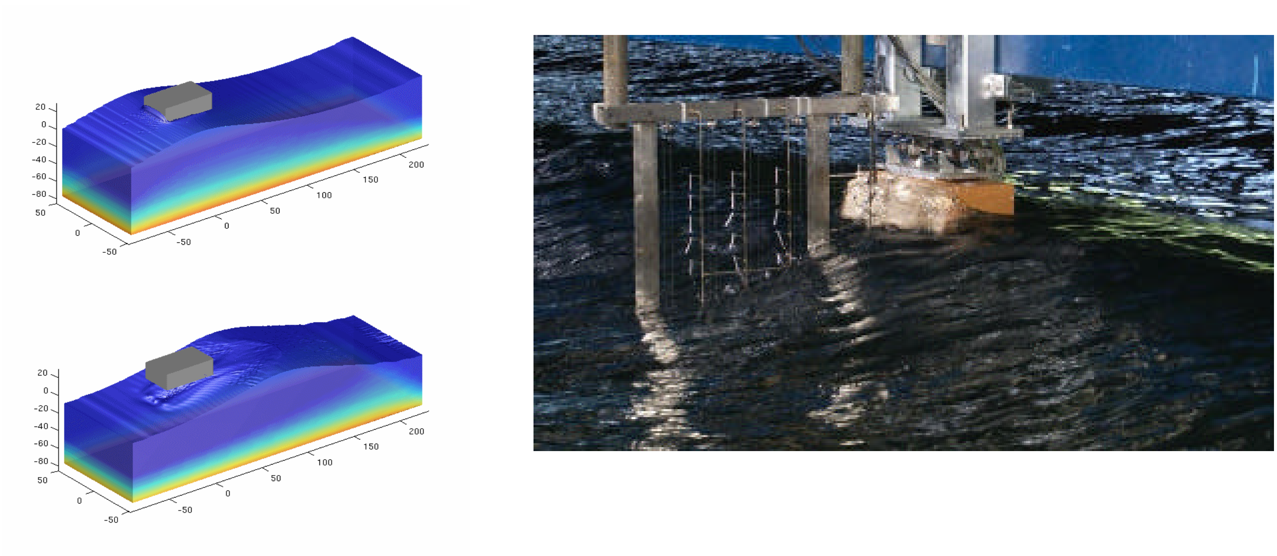


Figure 9-2: ComFLOW simulation and model test of a simplified wave in deck load

Details of this part of the project can be found in the WP2 Technical Reports. In this section a summary will be given of the work on the following subjects:

- Implementation of waves.
- Moving rigid bodies.
- Coupling of ComFLOW with diffraction results.
- Structural interface to Finite Element codes.
- Benchmarking and validation.

9.1.1 Implementation of waves

For the simulation of wave impacts on offshore platforms, waves have been implemented in ComFLOW, see Figure 9-3. The waves are initiated using Airy wave theory and 5th order Stokes theory. Velocities and wave height are calculated using the wave theory and are prescribed at the inflow boundary. When using linear theory, the velocities have been stretched by the Wheeler stretching method.

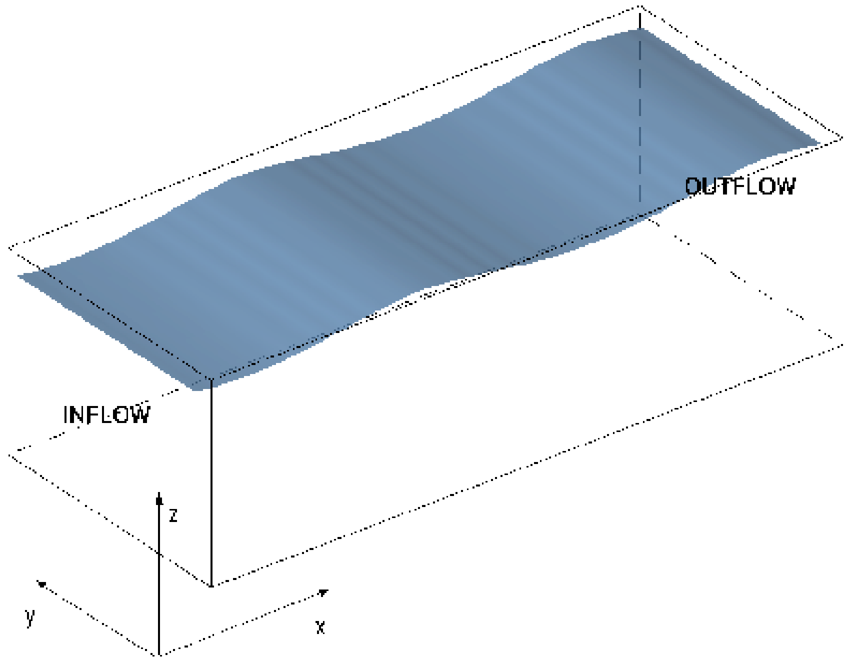


Figure 9-3: Wave simulation in ComFLOW

To prevent the waves from reflecting from the boundaries into the domain, a dissipation zone has been added at the end of the domain. The tangential sides of the domain are solid walls, which have been placed at such a distance that the simulation results are not influenced by them. To damp the waves in the dissipation zone, pressure damping has been used: a pressure term has been added at the free surface, that counteracts the wave motion.

When damping the wave completely, the water level was found to be rising. This was caused by the theoretical description of the wave: the net mass flux at the inflow boundary is positive. To overcome this problem, the solid wall in front of which the wave has been damped has been changed to an outflow boundary where a hydrostatic pressure is prescribed assuming the water height to be the initial calm water level. This way, the water is forced to flow out when the water level rises. This works nicely, the constant rise of the water level has no longer been observed. The exact amount of damping and reflection, using the dissipation zone, which is always in the range of a few per cent, has to be investigated further in future research.

At the free surface, boundary conditions for the velocity and pressure have to be prescribed.
The prescription of the velocities was found to have a very large influence on the wave simulations. When comparing the prescribed velocities with the theory, the best method for the wave simulations is to use a linear extrapolation of the velocities in the direction normal to the free surface. In the case of rather low waves, this direction will mostly be the negative z-direction. For a stable and robust discretisation, the idea of mass conservation in S-cells has been dropped. This does not violate mass conservation in the whole domain, because in S-cells no mass conservation is needed.

The developed procedure in SAFE-FLOW allowed an accurate description of the free surface, even in steep waves, see Figure 9-4 for an example.

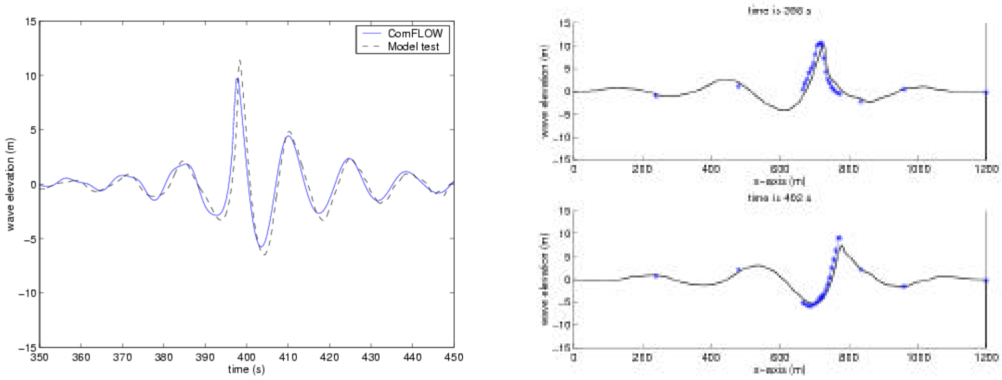


Figure 9-4: Simulation of steep waves in ComFLOW (dots and dashed line) and in the basin (solid lines)

To further test the wave propagation, two-dimensional simulations of regular and irregular waves have been performed. The results of the regular wave simulations have been compared with wave theories. The number of time steps in a period and the number of grid cells per wavelength and in the wave height have been varied which have shown to be of large influence. When using enough grid cells and time steps, the error in the simulations is less than 5% after a simulation time of 4 periods. This energy dissipation is probably due to the boundary conditions at the free surface and the displacement of the free surface. In a small domain with only a few periods of simulation the dissipation is so small, that the influence will not be severe in the simulations of local wave impact on offshore platforms.

The investigation of wave propagation has been concluded with simulations of wave loading on a SPAR platform, see Figure 9-5. Forces, pressures and wave heights have been calculated and compared to measurements. The results show that ComFLOW is able to reproduce results from the model tests with a satisfying accuracy, see Figure 9-6. Short waves are much more influenced by the SPAR than long waves which results in the need of a finer grid.

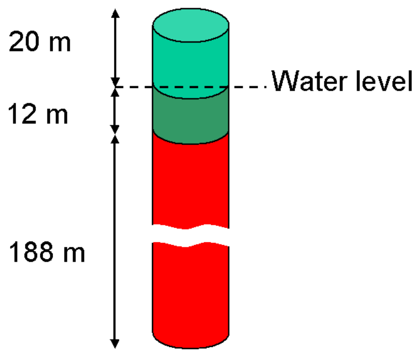


Figure 9-5: Segmented model of the SPAR

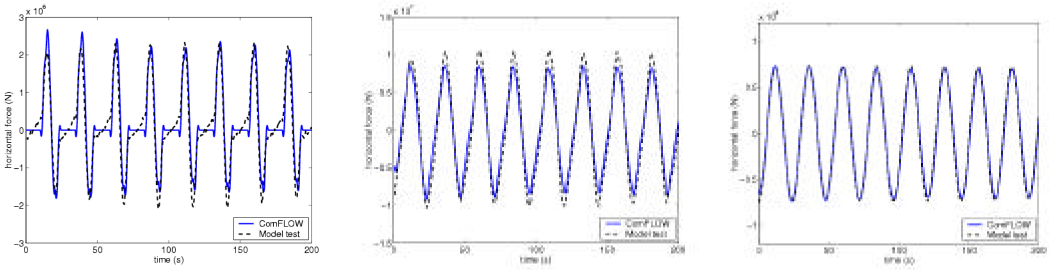


Figure 9-6: Measured and calculated loads on the upper segment (left), mid segment (middle) and lower segment (right)

9.1.2 Moving rigid bodies

The implementation of moving rigid bodies in ComFLOW has been successfully completed during the SAFE-FLOW project.

In the case that moving objects are present in the computational domain, the Navier-Stokes equations for the incompressible viscous fluid remain unchanged with the extra criterion that the fluid velocity at the boundary of the object is equal to the object velocity. The geometry of the moving object can have an arbitrary form, and is described by a finite element description. The moving body has to be displaced every time step according to a prescribed motion. Because a fixed Cartesian grid has been used, the piecewise linear geometry cuts through the cells. Volume and edge apertures, that describe the part of a cell and cell face respectively that is open for flow, are defined. Each time step, these apertures have to be recalculated. In three dimensions, it is very difficult to calculate the apertures exactly. In this approach, each time step an approximation has been made of the geometry. At the start of a computation, the moving object is covered with markers which form a kind of grid. The density of the markers should be at least the density of the computational grid, at the boundary the density should be somewhat higher (see Figure 9-7 for an example of such a grid). Around every marker, a virtual volume has been placed, such that the total object has been filled by the sum of the virtual volumes. Every time step when the object has to be displaced, the marker points and the virtual volumes are displaced. After that the volume apertures of a computational cell are calculated using the virtual volumes which cover the object. The edge apertures are calculated using a piecewise linear interface reconstruction method.

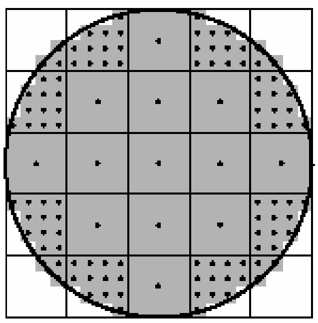


Figure 9-7: Example of a grid of marker points which cover a two-dimensional moving circle

The method is stable, also when very small cells occur because the object is moving through the fixed grid. In the calculation of the pressure, however, numerical spikes can occur. There are several reasons for the occurrence of the spikes. One of the reasons consists of a sharp corner of the moving object that causes a 'jump' in the edge apertures. These spikes have been suppressed, which results in a smooth signal for that particular case. Nevertheless, the other causes are still present, and should be solved in the future.

Figure 9-8 shows a validation case of simulation of green water on the deck of a moving FPSO. The motion of the FPSO is prescribed using experimental results from MARIN. Comparison of wave height, water height on deck and pressure signals with measurements are shown in Figures 9-9 and 9-10.

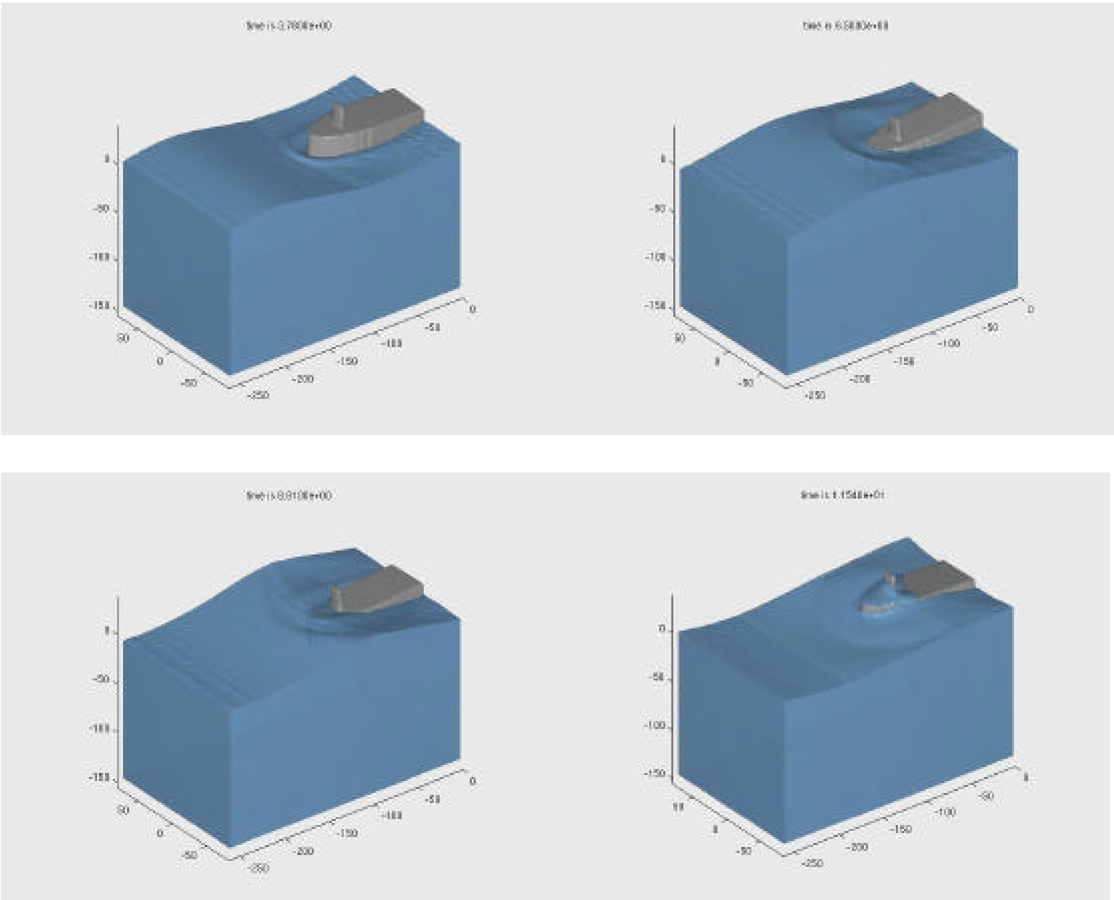


Figure 9-8: Simulation of motions of an FPSO bow with prescribed motions

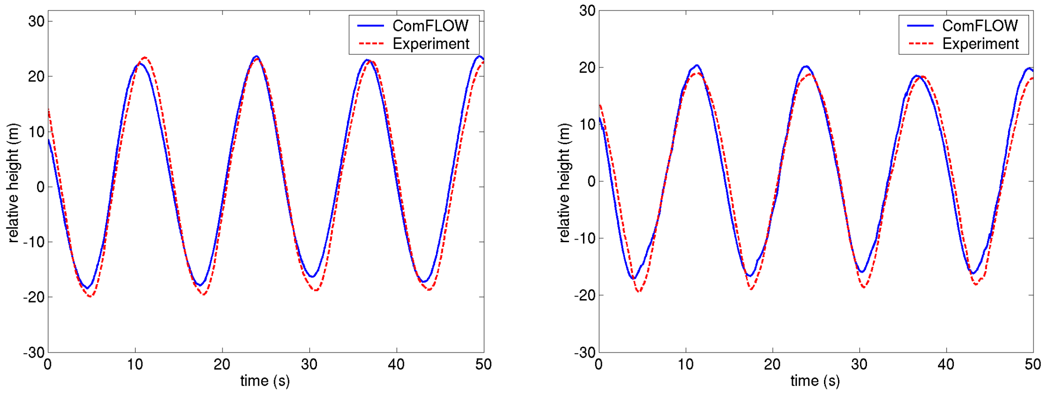


Figure 9-9: Relative wave height 30 metres (left) and 5 metres (right) in front of the FPSO

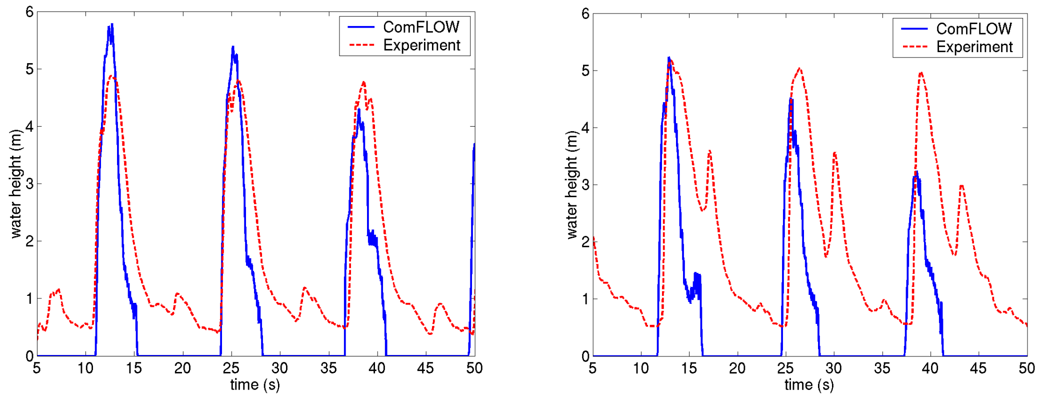


Figure 9-10: Water height on the deck close to the deck edge (left) and near the structure on the deck (right)

9.1.3 Coupling of ComFLOW with diffraction results

A coupling of ComFLOW to a diffraction code has been made with the following backgrounds (see Figure 9-11):

- ComFLOW is able to predict the high peak pressures due to bow slamming and the flow of green water on the deck, whereas the diffraction calculations cannot.
- The diffraction code is used for the calculation of the far wave field and the motion of the object because this is too time consuming for simulation in ComFLOW.

A critical event can chosen from the long time traces derived based on linear diffraction theory. This event will then be simulated with ComFLOW in a much smaller domain, using the diffraction results as initialisation, at the boundaries of the ComFLOW domain and for the prescription of the motion of the object.
To implement this coupling to a diffraction code, an interface has been created which transforms the diffraction data to a format which can be read by ComFLOW. In this interface, the fluid kinematics will be interpolated to the ComFLOW grid. Also a selection will be made of the frequencies which will be used in the simulation. Finally, an event will be selected from the generated time traces which will be simulated by ComFLOW.

The following steps are performed:

1. Use a diffraction calculation method to determine Response Amplitude Operators (RAOs) of fluid kinematics and ship motions in a large domain around the vessel.
2. Transfer the RAOs to time traces of ship motions, wave elevations and fluid velocities.
3. Select a critical event from these time traces.
4. Select a domain around the bow of the vessel in which the ComFLOW simulation will be done.
5. The fluid velocities and the ship position shortly before the critical event takes place are imposed on the grid of the local domain.
6. During the simulation, the fluid velocities need to be specified on the boundaries of the ComFLOW domain.

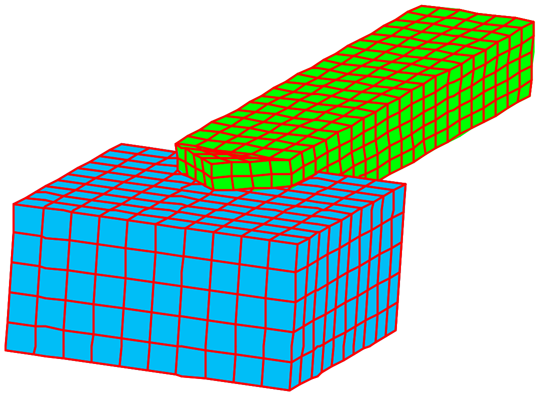


Figure 9-11: ComFLOW domain for local flow simulation (coupled to 3D diffraction analysis)

9.1.4 Structural interface to finite element codes

The SAFE-FLOW project also included a structural interface between ComFLOW and Finite Element Analysis (FEA) codes. The main objective of this particular task was to demonstrate the possibility of a two-stage analysis, where loads computed by the ComFLOW program are transferred into a Finite Element Analysis code. This is an important step, because this allows the incorporation of the complex pressure distributions calculated in ComFLOW to the detailed dynamic structural analysis of a structure.

The computational sequence for structural interface to the ComFLOW program can be described as follows:

1. Perform ComFLOW calculations for a chosen geometrical lay-out.
2. Create a Finite Element model of the structure. Consider various strengths of the panel and various boundary conditions for the structure.
3. Convert ComFLOW results, given as ASCII character files, into an appropriate Finite Element Analysis program input.
4. Perform non linear dynamics calculations to get response of the panel structure.

This procedure was demonstrated for a typical flat bow panel, see Figures 9-12 and 9-13.

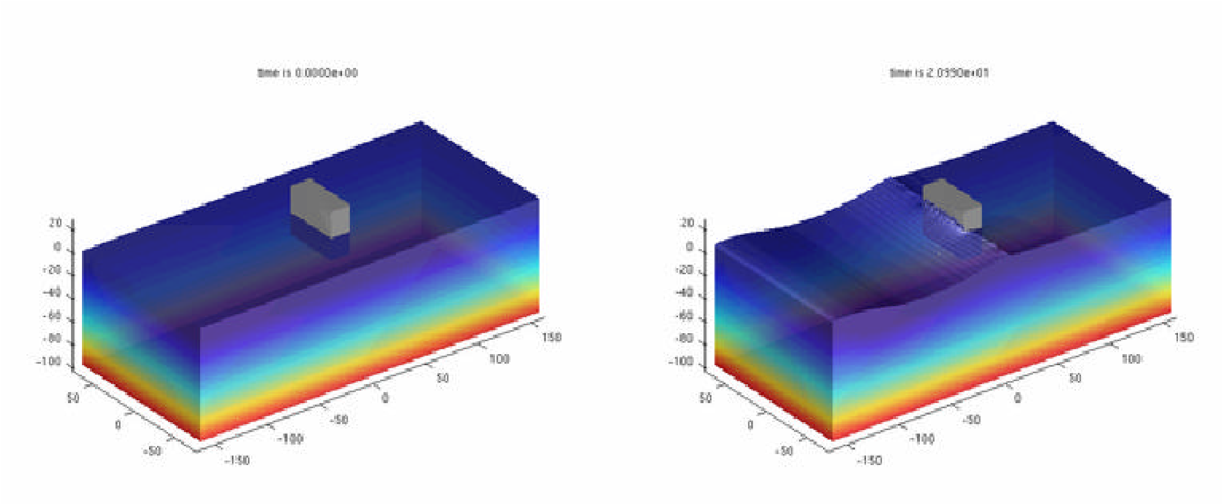


Figure 9-12: Wave load calculation in ComFLOW as input to FEA analysis

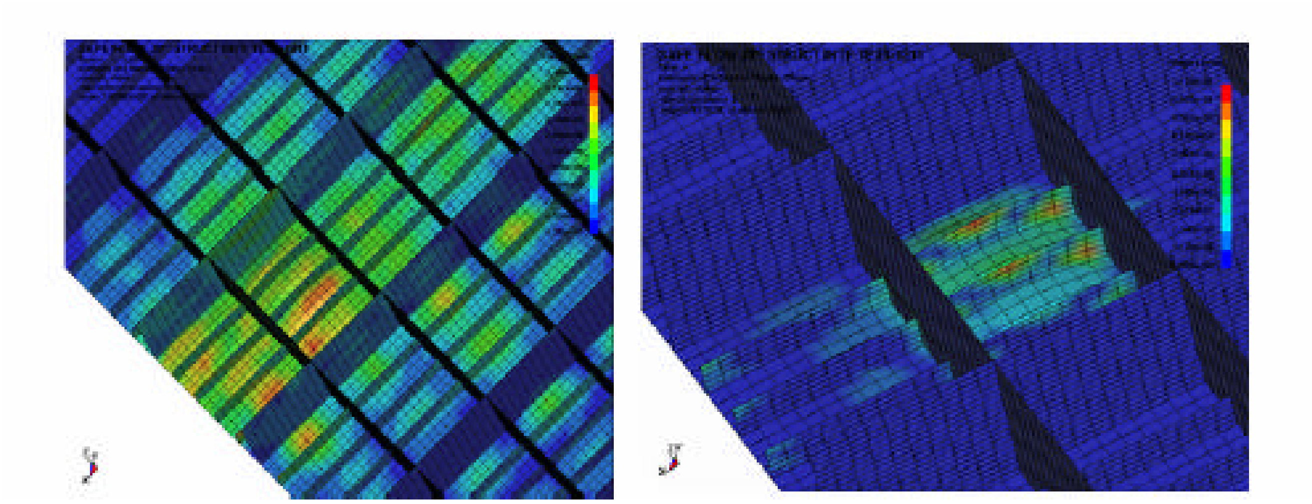


Figure 9-13: Resulting structural response in FEA code, allowing evaluation of dynamic stresses

9.1.5 Benchmarking and validation

During the SAFE-FLOW project a very extensive set of benchmarks and validations was carried out, based on test results available in general literature and based on specific tests carried out within SAFE-FLOW.

A special series related to tests performed in a dedicated dam breaking box, as shown in Figure 9-14. Water heights in the box were calculated, as well as the pressures and global loads on the structure in the flow and the back wall of the box.

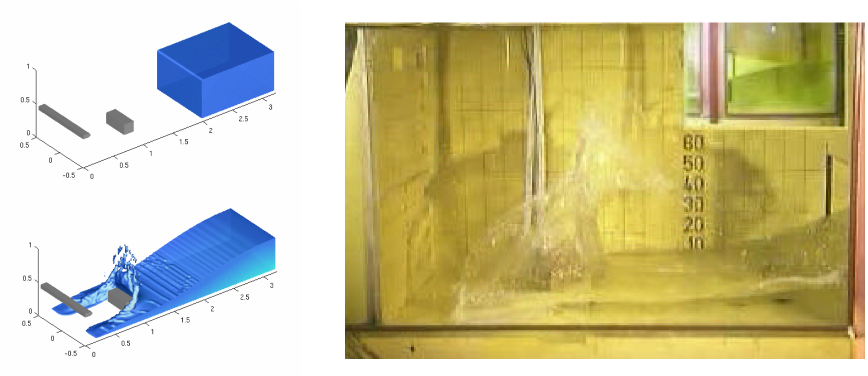


Figure 9-14: Simulation and model test with a special dam breaking box

Generally a very good agreement was found for the water heights as well as for the pressures and loads, see Figure 9-15 as example for the situation shown above. Local pressures need further investigation.

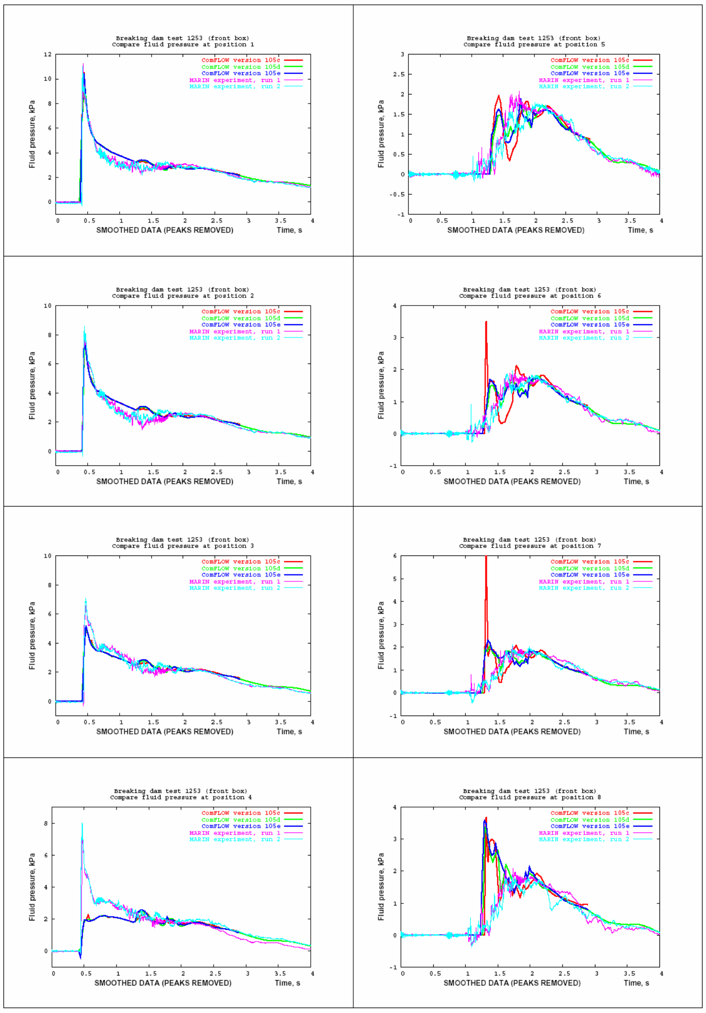


Figure 9-15: Comparison measured and calculated loads with different versions of ComFLOW

Also the important extension of ComFLOW with moving bodies has been tested extensively in the project. The following moving body benchmark cases have been calculated:

- two-dimensional wedge entry,
- two-dimensional cylinder entry,
- two-dimensional cylinder exit,
- three-dimensional cone entry,
- three-dimensional sphere entry.

Figure 9-16 shows a comparison between the two-dimensional wedge entry in ComFLOW and in literature.

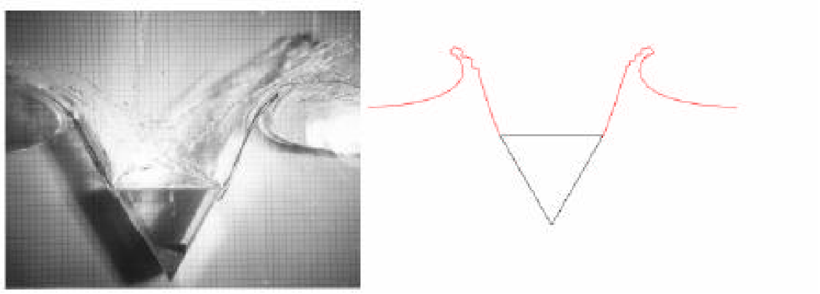


Figure 9-16: Comparison between the two-dimensional wedge entry in ComFLOW and in literature

As expected based on the physics of the problem, a nearly-perfect quadratic dependency of fluid force from the entry velocity was be observed for the cases tested, as shown in Figure 9-17 for the impact of a 2D cylinder in the fluid.

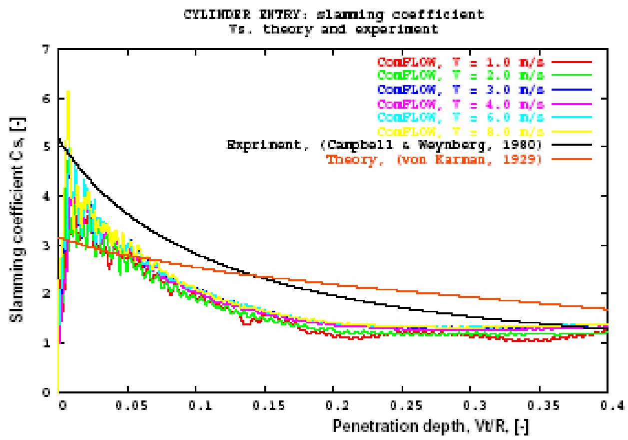


Figure 9-17: Comparison of slamming coefficients for a two-dimensional cylinder entry from literature and with ComFLOW for different entry velocities

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11 EXECUTIVE SUMMARY

The main objective of the SAFE-FLOW JIP was to develop guidance, calculation methods and risk assessment procedures for green water and wave impact loading ('water impact loading') appropriate to design at each stage of a floater project, namely: concept development or conversion specification, detailed design and operational assessment.

The SAFE-FLOW project focused on all aspects that are important for the problem: extreme waves, hydrodynamic impact loading and structural response. The aspects were placed in the framework of the development of a risk assessment model, design guidance and new standards (rules and regulations). For this purpose 4 main Work Packages and objectives were identified, which interacted as shown in the figure below:

WP1: WAVES	**WP2: HYDRODYNAMIC LOADING**	**WP3: STRUCTURAL**
Objective: Identification of critical wave (weather) conditions for wave impact problems and their physics and development of probabilistic wave models	Objective: Understanding of wave impact and green water physics, and development of methodology for the prediction of impact loading	Objective: Define a wave impact loading calculation methodology and propose an optimum structural design methodology (light and safe)
WP4: RISK & DESIGN Objective: Develop a risk-based methodology for wave impact loading issues in different phases of a floater project		

The achievements of the project can be summarised as follows:

- Within Work Package 1 (WAVES) methods were developed for the identification of critical wave conditions for wave impact problems and their physics. Together with WP2 the vertical free surface velocity was identified as the main factor in the wave impact problem. A probabilistic wave model was developed based on second order theory and compared to model test wave conditions and offshore measurements.

- Within Work Package 2 (HYDRODYNAMIC LOADING) a large step was made in the understanding of wave impact and green water physics. This resulted in the development of a completely new methodology (included in the 'BowLab' program) for the prediction of impact loading.

- In addition to that, a part of WP2 was dedicated to the modelling of water impact problems with Computational Fluid Dynamics (CFD) in the development of the ComFLOW program. Important progress was made in the fields of the implementation of waves, moving rigid bodies in the flow, coupling of ComFLOW with diffraction results and a structural interface to Finite Element Analysis codes. Also a lot of attention was paid to benchmarking and validation. The resulting ComFLOW program is unique in its capabilities of simulation complex wave loads on offshore structures.

- Within Work Package 3 (STRUCTURAL) structural models were defined appropriate to the assessment of green water loading and a wave impact loading and response calculation methodology. Results obtained suggest that ALS (Accidental Limit State) criteria will normally govern for green water problems due to the very onerous nature of rare extreme events. However, either ULS (Ultimate Limit State) or ALS conditions can govern for wave impact design, depending principally on the height of the target structure up the bow (higher structures are only reached by more severe events). The suggested methodology is therefore to adopt ALS design criteria throughout, but to check ULS conditions as well where these are likely to govern, based on typical 100-year return conditions and conventional load and material partial safety factors. The ULS check provides a more conventional check that safeguards against such conditions being more significant, but also provides an assurance that the structure will not require excessive maintenance and repair due to repeated moderate water impact events.

- Within Work Package 4 (RISK AND DESIGN) a risk-based methodology was developed for wave impact loading issues based on the input from all other Work Packages. Recommendations for a new rules and regulations based on the output of SAFE-FLOW has been provided.

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